DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

THE

BULLETIN

OF THE

BEACH EROSION BOARD

OFFICE, CHIEF OF ENGINEERS
WASHINGTON, D.C.

VOL. 5 JANUARY 1, 1951 NO.1

DEPARTMENT OF THE ARMY

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VOL. 5 NO. 1

BRITISH COAST PROTECTION ACT OF 1949

The purpose of the Coast Protection Act of 1949 is to amend the law relating to the protection of the coast of Great Britain against ercsion and encroachment by the sea, to provide for the restriction and removal of works detrimental to navigation, to transfer the management of Crown foreshore from the Minister of Transport to the Commissioners of Crown Lands, and for purposes connected with the foregoing. The more important parts and sections of the act are summarized in the following paragraphs.

Part I

Section 1. Coast protection authorities are established with powers and duties in connection with protection of land in their area as are conferred or imposed on such authorities by the act. The council of each Maritime county area shall be the coast protection authority for that area.

Section 2. Coast protection boards may be established where it appears expedient for the protection of land in any area. Such boards shall be the coast protection authority for that area. They shall consist of representatives of the council of every maritime county area any part which is within the area for which the board is constituted and may also include representatives of other affected bodies or persons.

Section 3. Any coast protection authority may concur with any one or more other coast protection authorities in appointing a joint committee of those authorities either for the aggregate of the areas of the authorities or for any part thereof.

Section 4. A coast protection authority is empowered to carry out such coast protection work within or outside its area as may appear to be necessary of expedient for the protection of any land in its area. An authority may acquire any land required for the purpose.

Section 5. A coast protection authority proposing to carry out any coast pretection work, other than maintenance or repair, shall publish notice of that proposal. Any person objecting to the proposal shall be granted a hearing, after which decision will be made by the Minister of Health as to approval, rejection or modification of the proposal. Urgently needed protective work may be carried out without publication of notice.

Section 6. Where it appears to a coast protection authority that persons interested in land to be protected should pay charges, the authority may prepare a plan known as a "works scheme" which shall indicate the nature and estimated cost of the proposed work.

Section 7. Coast protection charges shall be levied against land benefiting by the work, as indicated by the works scheme. The charges shall not exceed the amount by which the value immediately after completion of the works is greater than the value would then be if the works had not been undertaken; the works in the future to be maintained without expense to person interested in the land.

Section 8. A coast protection authority shall publish notice of the preparation of the scheme and serve copies of the scheme on owners and occupiers of affected land and on persons on whom charges will be levied. Objectors shall be granted a hearing after which decision will be made by the Minister of Health as to approval, rejection or modification of the scheme.

Section 9. After approval of a works scheme by the Minister of Health, the authority is empowered to take all necessary steps for carrying out the work. Land owners have the option of carrying out the work themselves upon notice to the authority and subject to certain time limitations.

Section 10. Coast protection charges levied upon a person under a works scheme become due on completion of the work and service upon him of a notice specifying the amount of the charge, or in case of dispute, when the dispute is finally determined. Coast protection authorities may declare charges payable in installments covering not more than 30 years with interest at such rate as may be determined by regulations prescribed by the Minister of Health.

Section 12. A coast protection authority may serve notice of needed repairs to protective works on the owner or occupier of land. If the repairs are not completed within the specified period or if the repairs are urgently needed, the authority may take all necessary steps for carrying out the work.

Section 13. The reasonable cost of maintenance accomplished by an authority under section 12 may be recovered from the owner of the land, except for works constructed, altered or improved under a works scheme.

Section 14. A coast protection authority may be authorized to acquire by compulsory purchase any land which it is authorized to acquire under section 4 of the Act.

Section 16. Any person desiring to carry out coast protection work, other than maintenance or repair, must secure the consent of the coast protection authority in whose area the work is to be carried out. Works constructed, altered or improved without such consent, or in contravention of any conditions subject to which the consent was granted, may be required to be removed or altered.

Section 18. The excavation or removal of any materials on or forming part of the seashore is unlawful except under license granted by the coast protection authority for the area, subject to such conditions as it may determine.

Section 19. Where the performance of coast protection work by a coast protection authority results in depreciation of land, compensation therefor is authorized subject to certain conditions.

Section 21. The Minister of Health may make grants toward any expenditure incurred under the Act by a coast protection authority, subject to such conditions as may be determined by the Treasury.

Part II

Part II contains provisions for the safety of navigation. Consent of the Minister of Transport must be secured to construct, alter or improve works, deposit or remove objects or materials from the seashore so that obstruction or danger to navigation is caused or is likely to result.

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GENERALIZATION OF THE FORMULA FOR CALCULATION OF ROCK FILL DIKES AND VERIFICATION OF ITS COEFFICIENTS

by
Ramon Iribarren Cavanilles
with the collaboration of
Casto Nogales y Olano
Highway Engineers

Published in the Review of Public Works, May 1950, Madrid, Spain, as Generalizacion de la formula para el calculo de los diques de escollera y comprobacion de sus coeficientes, por Ramon Iribarren Cavanilles con la colaboracion de Casto Nogales y Olano, Ingenieros de Caminos; publicado en la Revista de Obras Publicas de Mayo de 1950, Madrid, Espana.

FOREWORD

The following translation is related to an article by the same author which appeared in translation in the Bulletin of the Beach Erosion Board, Vol. 3, No. 1, January 1949. It is published at this time for the benefit of American engineers concerned with breakwater, jetty, and groin design and as a means of acquainting them with Mr. Iribarren's latest thinking on the subject. The editors of the Bulletin wish to acknowledge the kindness of Mr. R. O. Eaton, through whose efforts the translation was made available.

The opinions expressed are those of the author and not necessarily those of the Beach Erosion Board.

In the booklet Una fórmula para el cálculo de los diques de escollera (A Formula for the Calculation of Rock-fill Dikes), published July 1938* there was determined the expression

$$P = \frac{N\ A^3\ d}{(\cos\alpha - \sin\alpha)^3\ (d-1)^3}$$

where

P = weight of the individual stones or blocks in kilograms,
N = 15 for dikes of natural rock fill,
N = 19 for dikes of artificial block fill,
$A = 2h$ = total height of the wave that breaks on the dike, measured in meters

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*Translated and published in the January 1949 Bulletin of the Beach Erosion Board, Office, Chief of Engineers, Department of the Army. The coefficient K of that booklet is denoted by N in this one, to avoid confusion with $K = \coth \pi \frac{H}{L}$, and similarly, the angle a we here denote α. Translation was made by D. W. Hullinghorst.

d = specific weight of material of the stones in tons* (metric) per cubic meter

α = angle of the dike's side slope with the horizontal.

Before the preliminary determination of those tentative values of N, each based only on a single observed case - the natural rock fill dike of Orio and the artificial rock fill dike of San Juan de Luz - the booklet also stated:

"It only remains now to determine the coefficient N and verify if it is sensibly constant, as seems to follow from the material presented, or varies with the other elements of the formula.

"In the worst case, a coefficient similar to the classic and variable coefficient C of the formula of uniform flow, $V = C\sqrt{R\,i}$, will be considered."

In spite of the twelve years intervening, during which the reasoning followed for the deduction of the formula has been refined, in a manner that might have been advantageously taken into account by the recent translators of the booklet, the coefficients 15 and 19 still stand, due to the satisfactory results always obtained in numerous cases of practical application.

The formula is actually derived for the upper slope of the dikes, and a generalization for all the depths of the work was indicated only tentatively at the end of the booklet. In this connection substantially the following was said:

"This generalization of the formula assumes a certain margin of security, but it is not logical to apply on the sea the strict results obtained from theoretical formulas when on land it is usual to multiply them by ample safety factors.

"I should be most grateful to my colleagues who, acquainted in detail with concrete practical cases, would kindly furnish me information for refining the coefficients."

Unfortunately those necessary details of each particular case, and especially the damages experienced, are difficult to obtain. Therefore, in this study, we are going to make special mention of one of singular interest.

We refer to the interesting compilation on the port of Argel concerning weights of stones or blocks and their corresponding stable slopes at various depths, after repairs of numerous damages to deficient slopes. This outstanding compilation was made by

*In this paper, the ton unit is the metric ton of 1000 kilograms = 2205 lbs. - Trans.

Messrs. J. Larrás and H. Colin in their article of December 1947 published in the periodical Travaux.

The following data are obtained from page 609 of that compilation showing the corresponding slopes, depths, and weights of blocks or stones.

Materials	Minimum weight, metric tons	Maximum slope	Minimum Depth, meters
Artificial blocks	50	5/4	-5
Natural stones	4	3/2	-11
same	1	3/2	-14
Quarry waste	-	2/1	-18

There can also be adopted, as a stable upper surface slope, that of 3/1 formed by 50-ton concrete blocks, adopted for strengthening the North dike which, according to the cited article, has withstood perfectly even the worst storm (3 February 1934) ever suffered by the port of Argel and which destroyed a large part of the Mustafa dike. Likewise, we can adopt the toe depth of 35 meters, which this North dike reaches.

The maximum characteristics of waves from that very violent storm are known with an approximation acceptable for practical purposes, and will be used in comparisons which follow. The wave period reached 13 3/4 seconds.* The maximum amplitude of a beacon buoy** in the exterior of the port was 7 meters, and this constitutes an approximate upper limit to the wave height.

To explain the 9-meter sheets observed passing over the parapets of the dikes, it has been conjectured, simply by inertia, and abstracting other forces such as those of buoyancy, that the amplitude of the vertical movement of the buoy was less than the wave height.

The buoy and whole float of constant horizontal section can be likened to a vertical oscillator whose own natural period, was undoubtedly much less than the 13 3/4 seconds of the period of the wave. Under these conditions, the maximum vertical amplitude of the oscillator, or buoy, must be practically equal to or, better, somewhat greater than that of the sea.

For hypothetical lesser periods of the sea, that might approximate the buoy's own natural period the vertical oscillations of the buoy, amplified by resonance, would be much greater than

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* See the view presented to the second question (subject) of the second section of the XVI International Congress of Navigation by Messrs. M. Benezit and M. Renaud.

**"boya de balizamiento"

periods even briefer than the buoy's own period, could the buoy's oscillations be less than those of the sea.

For this to have happened, during the observations of the storm of Argel to which we refer, it should have been necessary that the natural vertical period of the buoy itself be significantly greater than the 13 3/4 seconds, which we estimate to be impossible in a buoy of that type.*

The maximum height of wave before reaching the dikes was, consequently, approximately 7 meters or somewhat less, and only on breaking over the dikes did this height, amplified** by the works (structure) itself, reach the order of 9 meters, as is demonstrated below.

Due to the steep side slopes of the rock fill dikes, or rather, the short distance (relative to wave length) which the wave traverses from the vertical line through the dike toe to the highest point reached on the dike, the energy loss must be small, in spite of the rock fill roughness, always small relative to the wave length. Moreover, even were the energy loss to be appreciable, disregarding it only augments correspondingly and conveniently the margin of safety.

In a sense (limited, for a breakwater is not a beach although the effects resemble one another for waves of the length under consideration), the graphs of our article published in the Revista de Obras Públicas last November, in which the curves corresponding to slopes exceeding 10 per cent are almost coincident with those from conservation of energy, may serve to verify the smallness of such energy loss.

More authentic verification, of this and other simplifying assumptions that are necessitated in these complex subjects, is the comparison with actual observations on the dikes of Argel, themselves, which we make in the following.

* "Boya de balizamiento"

** Inclosed in vertical dikes, a similar amplification is produced by the structure itself--amplification which in this case makes the amplitude of the vertical surface movement, on the wall, generally exceed twice the wave height.

See the booklet Cálculo de diques verticales (Calculation of Vertical Dikes), also published in July 1938 and translated and published in the Bulletin of the International Navigation Congress, July 1939. Also see the experimental confirmation of said amplification, constituted by Figure 13 and 18 of the article of A. Stucky and D. Bonnard, published in Travaux of January 1937. The results obtained in those publications are likewise admissibly proportioned to the height of wave indicated and its corresponding amplification.

If we designate by H_p the depth of the toe of the slope and by L_p, h_p, K_p the characteristics* of the wave over the depth H_p; by H the depth of the point on the slope that we are considering, and by L, h, K, etc., the corresponding characteristics of the wave, and if, for the reasons presented, we admit the conservation of energy, one deduces immediately:

$$h = h_p \frac{p}{p_p}$$

where

$$p = \sqrt{K}\sqrt{\frac{a_o}{a}} \quad \text{and} \quad p_p = \sqrt{K_p}\sqrt{\frac{a_o}{a_p}},$$

deduced from the approximate curve

$$\frac{h}{h_o} = \sqrt{K}\sqrt{\frac{a_o}{a}}$$

of figure 22 of the cited report, or from its corresponding table, pages 32 and 33.

The approximate curve can be utilized, neglecting the breaking factor $\sqrt{\frac{b_o}{b}}$ because the action of breaking is so very rapid that there is insufficient time for its effects to be really produced before breaking, and even if this should not be entirely so, as the said effects are very similar for all breakings, on being neglected they would remain implicitly in the coefficients N, determined by direct observation of the phenomena.

With respect to the generalization of the formula, mentioned at the beginning of this study, it is proper to note that for some years it has been refined on the basis of the following reasoning; and that, although this reasoning also supposes some simplifications, the final result we get will confirm, on comparison with an actual case (always the final test in technology), that the degree of approximation is also sufficient for practical applications.

The maximum horizontal particle velocity of the wave (on breaking under its limiting conditions, $\nu_h = \sqrt{gh}$,**) being the principal cause of the removal of the stones of the breakwaters, the effects of the maximum orbital velocity, also horizontal, corresponding to any depth

$$\nu_{max} = \frac{\pi r}{T},$$

*Both these notations and the others can be found in our report presented to the fourth communication of the Second Section of the XVII International Congress of Navigation.

**See the cited booklet of July 1938, or its translation, and the aforementioned report to the Lisbon Congress.

will be similar to those of a hypothetical, or virtual, wave of equal velocity. That is, the equality $v_{max} = v_h$ would have to hold, or in orther words,

$$\frac{\pi r}{T} = \sqrt{gh'} = \sqrt{g \frac{A'}{2}} \quad .$$

The semiperiod being

$$T = \sqrt{\frac{\pi}{g} \cdot L \cdot K},$$

immediately one obtains the height of the virtual wave:

$$A' = 2\,h' = \frac{2\,\pi\,r^2}{LK} \quad ,$$

which, introduced in place of A in the formula, thus generalizing it, permits us to calculate the slope or weight of the stones at the depth under consideration.

Likewise, since

$$K = cth\,\pi \frac{H}{L}\ , \ LK = Lo,$$

and r in contact with the slope face

$$r = r_f = \frac{h}{Sh\,\pi \frac{H}{L}} \quad ,$$

one obtains

$$A' = \frac{2\,\pi\,h^2}{Lo\ Sh^2\,\pi \frac{H}{L}} \quad .$$

The inclination of the slope, and other circumstances, could influence all this somewhat, but not varying much from one dike to another, its influence would also remain implicitly in the coefficients which, as has been indicated, are determined by direct observation.

As confirmation of all that has been presented, and has been applied for several years on numerous works that have resisted satisfactorily violent storms, we are going to apply it to the authentic and interesting case of the dikes at Argel on which for the reasons presented, we can soundly begin with:

$H_p = 35$ m.; $A_p = 2\ h_p = 7$ m.,

$$2\ L_o = 1.56\ (2T)^2 = 1.56 \times 13.75^2 = 295 \text{ m}; \quad L_o = 147.5 \text{ m}.$$

For various values of the depth H, measured from the mean level of movement, we obtain the information shown in Table 1.

The curves of variation of A' and 2h as functions of the depth H, referred to the mean level of movement, are shown in Figure 1, in which also are drawn the curves of surface super-elevations S_h and the line of theoretical breaking H = h, corresponding to steep slopes of the rock-fill dikes.*

The point of intersection of this straight line with the curve 2h determines for us the height of 9.70 meters of the wave on breaking over the dike, approximating, though somewhat greater than, the 9 meters adopted by the Argel engineers.

In Figure 2 are shown the wave heights 2h referred to the level of the calm sea, $H_r = H = S_h$, the superelevations S_h taken into account, and the corresponding curve that gives us the virtual heights A', which we must take into account for the calcuation of the dike in its submerged zones.

The type section of the dikes of Argel, sanctioned by very long experience and deduced from the interesting table compiled by MM. Larrás and Colin, is that indicated by the solid line in Figure 3.

If, in a given zone, the slope of a dike is steeper than that required for equilibrium or stability, settling will result during storms, which, flattening the slope, will tend to adapt it to said equilibrium. It is, therefore, logical to make the corresponding confirmations, determining by means of theoretical calculations and the coefficients which it is desired to compare with actuality, the slopes corresponding to the points or zones whose characteristics are known through prolonged direct observation.

For this, from the formula

$$P = \frac{N\ A^3\ d}{(\cos\alpha - \sin\alpha)^3\ (d-1)^3}$$

* Note the section h, Rotura de las olas (Breaking of the Waves) page 26 et seq., of the cited report presented to the XVII International Congress of Navigation, convened in Lisbon.

Table I

H	35	31,5	28	24,5	21	17,5	14	10,5	7	3,5	0
$\frac{H}{L_0}$	0,2373	0,2136	0,1898	0,1661	0,1424	0,1187	0,0949	0,0712	0,0475	0,0237	0
$K = \mathrm{Cth}\,\pi\,\frac{H}{L}$	1,3231	1,3751	1,4382	1,5171	1,6160	1,7460	1,9300	2,1964	2,6548	3,7128	∞
$2\,L = 2\,L_0 \cdot \frac{1}{K}$	222,96	214,52	205,03	194,49	182,55	168,92	151,75	134,31	111,13	79,47	0
$P = \sqrt{K} \cdot \sqrt{\frac{a_0}{a}}$	0,9211	0,9277	0,9372	0,9505	0,9690	0,9951	1,032	1,088	1,181	1,379	∞
$2\,h = 2\,h_p \cdot \frac{p}{p_p}$	7,00	7,05	7,12	7,22	7,36	7,56	7,84	8,27	8,98	10,48	∞
$A' = \dfrac{2\,\pi\,h^2}{L_0\,\mathrm{Sh}^2\,\pi\,\frac{H}{L}}$	0,39	0,47	0,58	0,72	0,93	1,25	1,78	2,78	5,20	14,96	∞
$S_h = \dfrac{\pi\,h^2\,K}{2\,L}$	0,23	0,25	0,28	0,32	0,38	0,46	0,61	0,88	1,52	4,03	∞
$H_r = H - S_h$	34,77	31,25	27,72	24,18	20,62	17,02	13,39	9,62	5,48	— 0,53	$-\infty$

H	35	31,5	28	24,5	21	17,5	14	10,5	7	3,5	0
$2\,h = 2\,h_p \cdot \frac{p}{p_p}$	6,45	6,49	6,56	6,65	6,78	6,96	7,22	7,62	8.27	9,65	∞
$A' = \dfrac{2\,\pi\,h^2}{L_0\,\mathrm{Sh}^2\,\pi\,\frac{H}{L}}$	0,33	0,40	0,49	0,61	0,79	1,06	1,51	2,36	4,41	12,69	∞
$S_h = \dfrac{\pi\,h^2\,K}{2\,L}$	0,20	0,21	0,24	0,27	0,32	0,39	0,52	0,75	1,29	3,42	∞
$H_r = H - S_h$	34,80	31,29	27,76	24,23	20,68	17,11	13,48	9,75	5,71	0,08	$-\infty$

Table 2

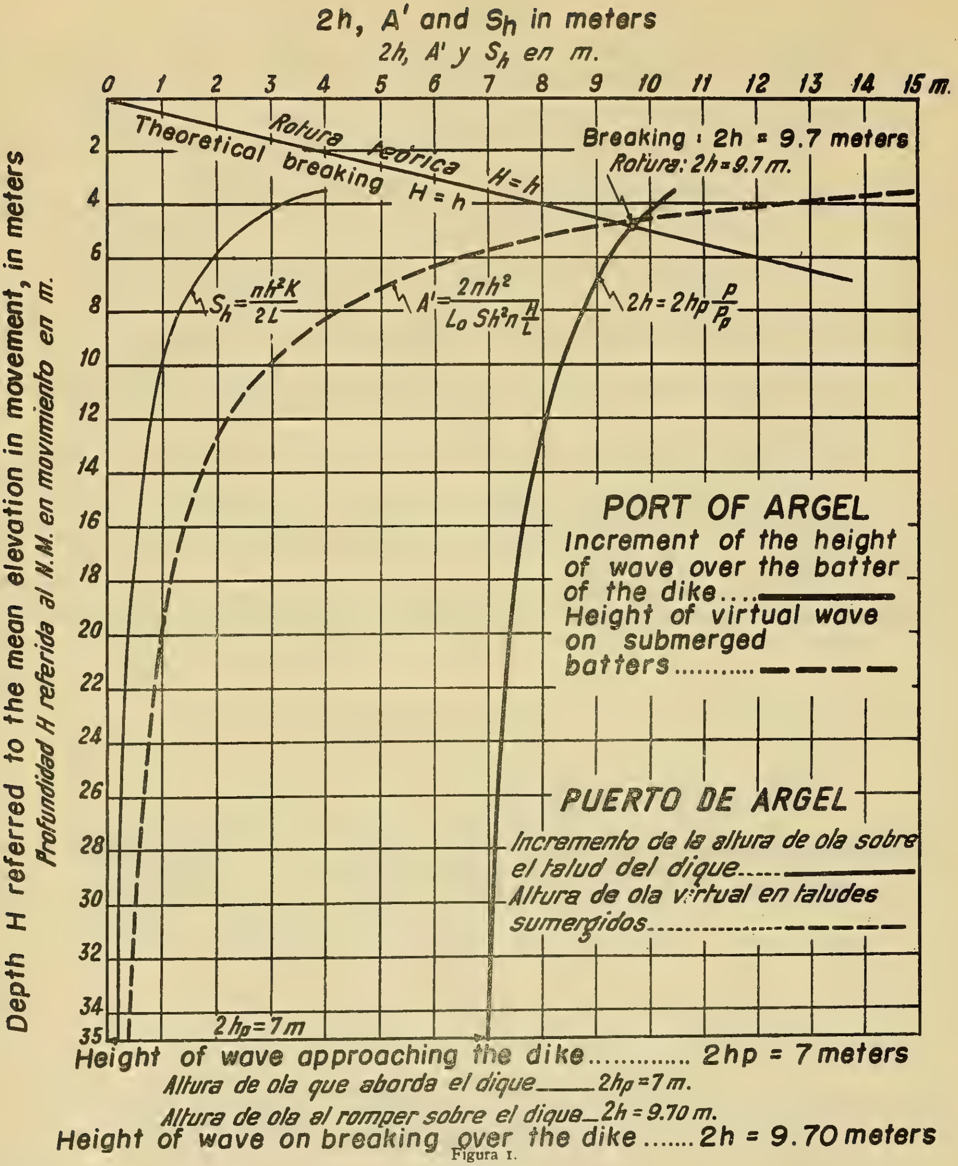


Figura I.

Figure I

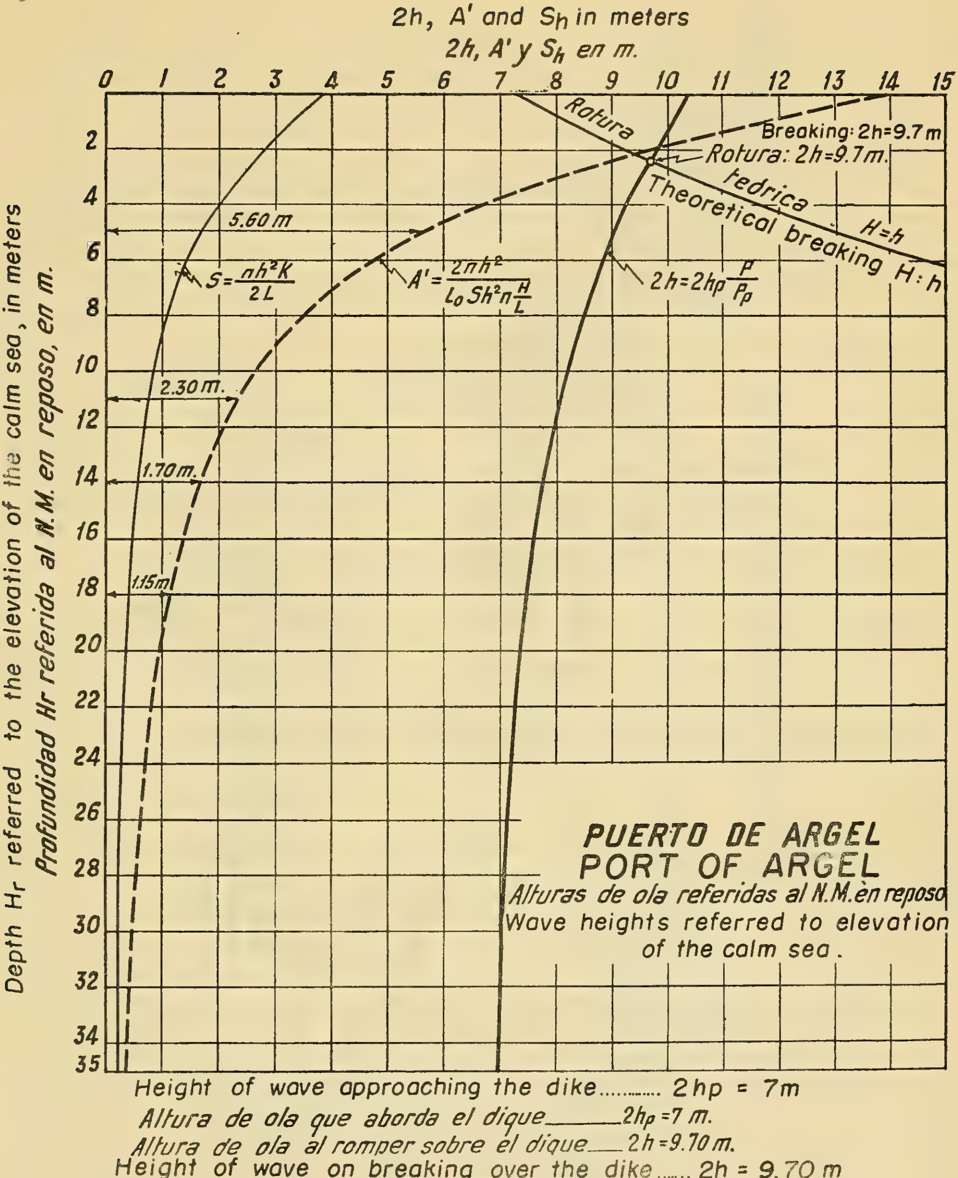


Figura 2.

Figure 2

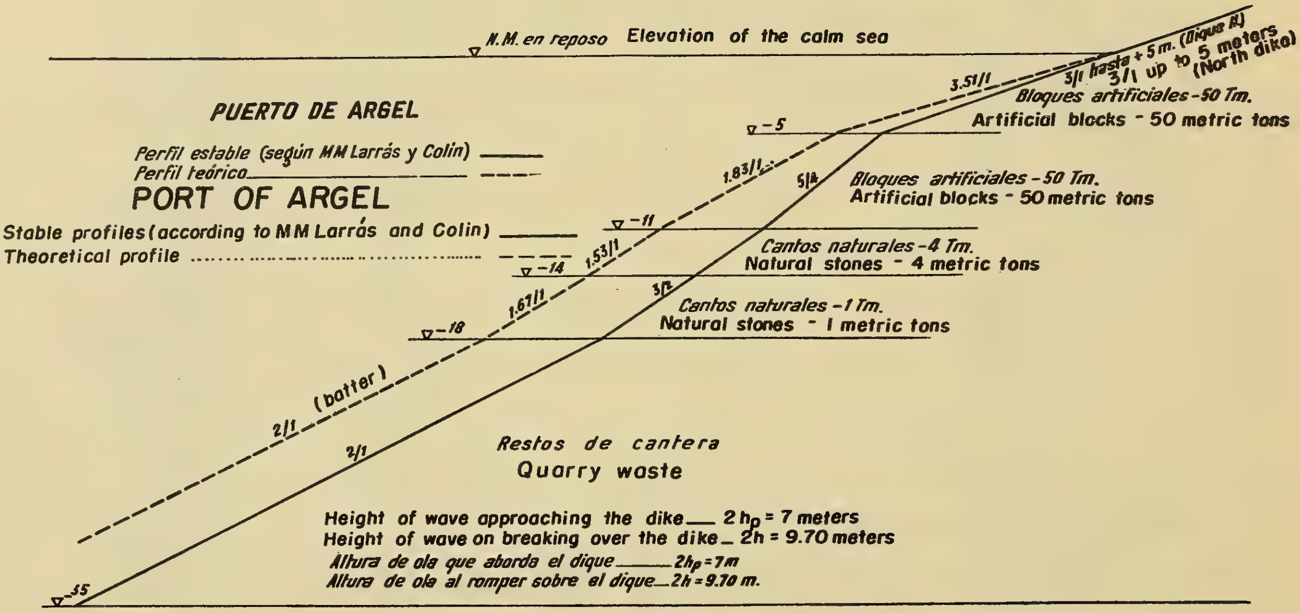


Figura 3.

Figure 3

we deduce

$$\cos \alpha - \sin \alpha = \frac{ct\,\alpha - 1}{1 + ct^2 \alpha} = \frac{A}{d-1} \sqrt[3]{\frac{Nd}{P}}$$

from which, the last member being known, immediately we deduce the batter $ct\,\alpha$ that enables us to draw the theoretical profile, in order to compare it in Figure 3 with that determined by nature.

Thus, we obtain for the batter of the upper 50 ton* concrete blocks:

$$\frac{ct\,\alpha - 1}{\sqrt{1 + ct^2 \alpha}} = \frac{9.7}{1.36} \sqrt[3]{\frac{19 \times 2.36}{50,000}} = 0.688$$

whence the batter $ct\,\alpha = 3.51$

For the rock fill of artificial blocks situated below the depth $H_r = -5$ meters, referred to the calm level, we obtain

$$\frac{ct\,\alpha - 1}{\sqrt{1 + ct^2 \alpha}} = \frac{5.6}{1.36} \sqrt[3]{\frac{19 \times 2.36}{50,000}} = 0.397$$

whence the batter is $ct\,\alpha = 1.83$.

For the natural rock fill situated below $H_r = -11$ m:

$$\frac{ct\,\alpha - 1}{\sqrt{1 + ct^2 \alpha}} = \frac{2.30}{1.7} \sqrt[3]{\frac{15 \times 2.7}{4000}} = 0.293 ,$$

whence the batter is $ct\,\alpha = 1.53$.

For that situated below $H_r = -14$m:

$$\frac{ct\,\alpha - 1}{\sqrt{1 + ct^2 \alpha}} = \frac{1.70}{1.7} \sqrt[3]{\frac{15 \times 2.7}{1000}} = 0.343$$

or the batter is $ct\,\alpha = 1.67$.

For the deep zone $-H_r \geq 18$ m, composed of the quarry waste whose weight will vary from 1000 kilograms to a small minimum weight, the weight of the stones obtained for the fixed batter, $ct\,\alpha = 2$, would be

$$P = \frac{15 \times 1.15^3 \times 2.7}{\left(\frac{1}{\sqrt{5}}\right)^3 \times 1.7^3} = 143 \text{ kg},$$

* Metric ton, 1000 kg = 2205 lbs.

very acceptable for a weight of stones that, as has been indicated can range downward from 1000 kg to a small weight, and whose individual stones of weight less than 143 kg placed near the upper part would descend down the slope, so that by this process, and surely without subsidence of importance, the surface would become constituted of stones of greater weight.

The necessary weight of stones at the foot of the dike, where the height of virtual wave is 0.39 m, would be only

$$P = \frac{15 \times 0.39^3 \times 2.7}{\left(\frac{1}{\sqrt{5}}\right)^3 \times 1.7^3} = 5.6 \text{ kg.}$$

The theoretical profile is obtained in this way and is indicated by the broken line in Figure 3, whose approximation to the real profile is already very satisfactory. But, if instead of adopting as height of incident wave its upper limit $H_p = 7$ m, deduced from oscillations of the buoy, we assume the maximum height of wave of 9 m adopted by the engineers of Argel, we obtain Table 2, similar to the preceding and corresponding to a height $2\ h_p = 6.45$ m, determined after several trials, so that the maximum height of wave on breaking should be the 9 m cited.

Figure 1' similar to Figure 1, confirms that, in effect, the height of wave at breaking is 9.05 meters, that is, practically the same as 9 m.

In Figure 2' similar to Figure 2, are determined the height of virtual waves, A', corresponding to the various depths, in which, again by analogous procedure, we obtain the following batters;

For the surface slope of 50 ton concrete blocks:

$$\frac{ct\,\alpha - 1}{\sqrt{1 + ct^2\alpha}} = \frac{9.05}{1.4}\sqrt[3]{\frac{19 \times 2.4}{50000}} = 0.641$$

whence, the batter should be $ct\,\alpha = 3.04$

For the rock fill of artificial blocks situated below the depth $H_r = -5$m:

$$\frac{ct\,\alpha - 1}{\sqrt{1 + ct^2\alpha}} = \frac{5.0}{1.4}\sqrt[3]{\frac{19 \times 2.4}{50,000}} = 0.346$$

whence the batter should be $ct\,\alpha = 1.68$

For the natural rock fill situated below $H_r = -11$ m;

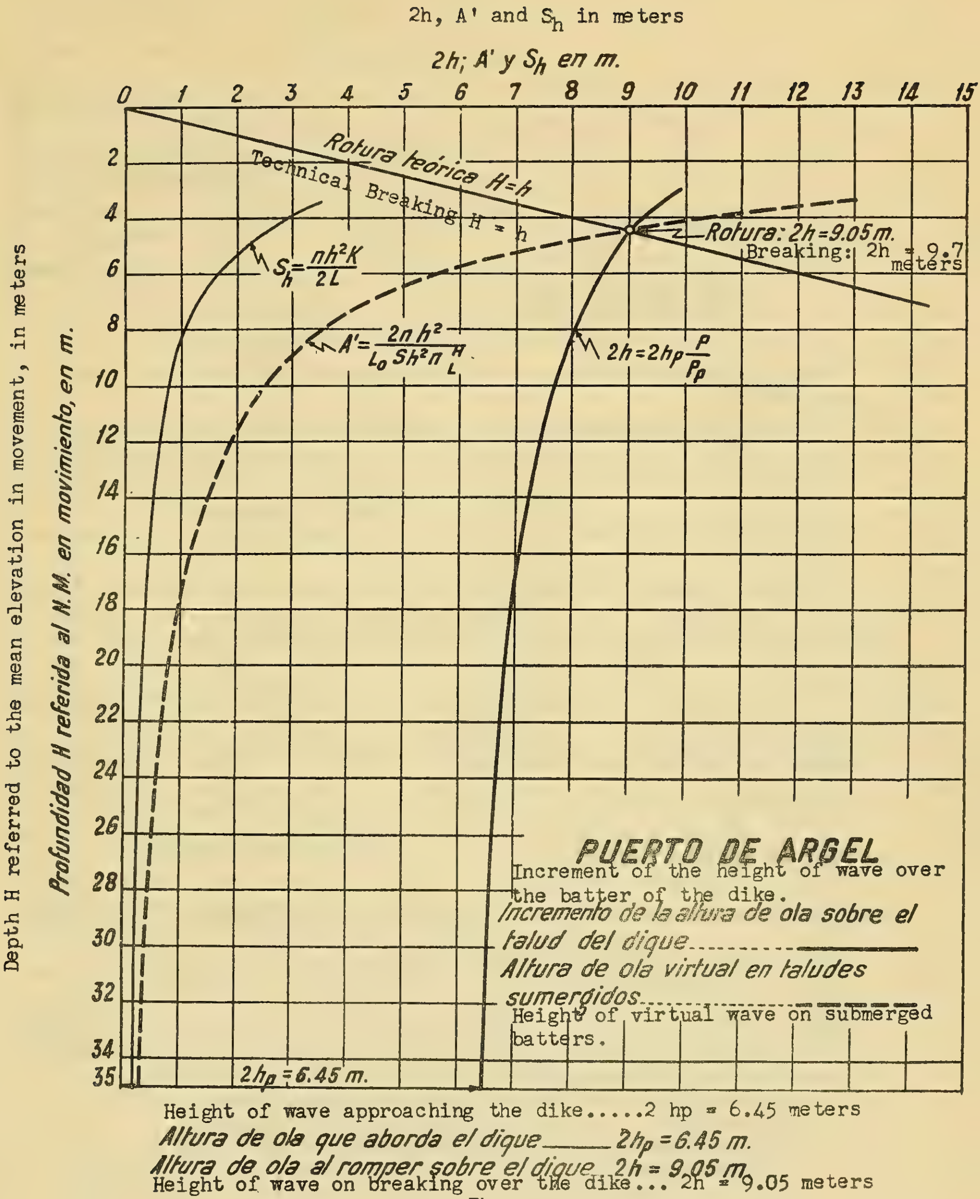
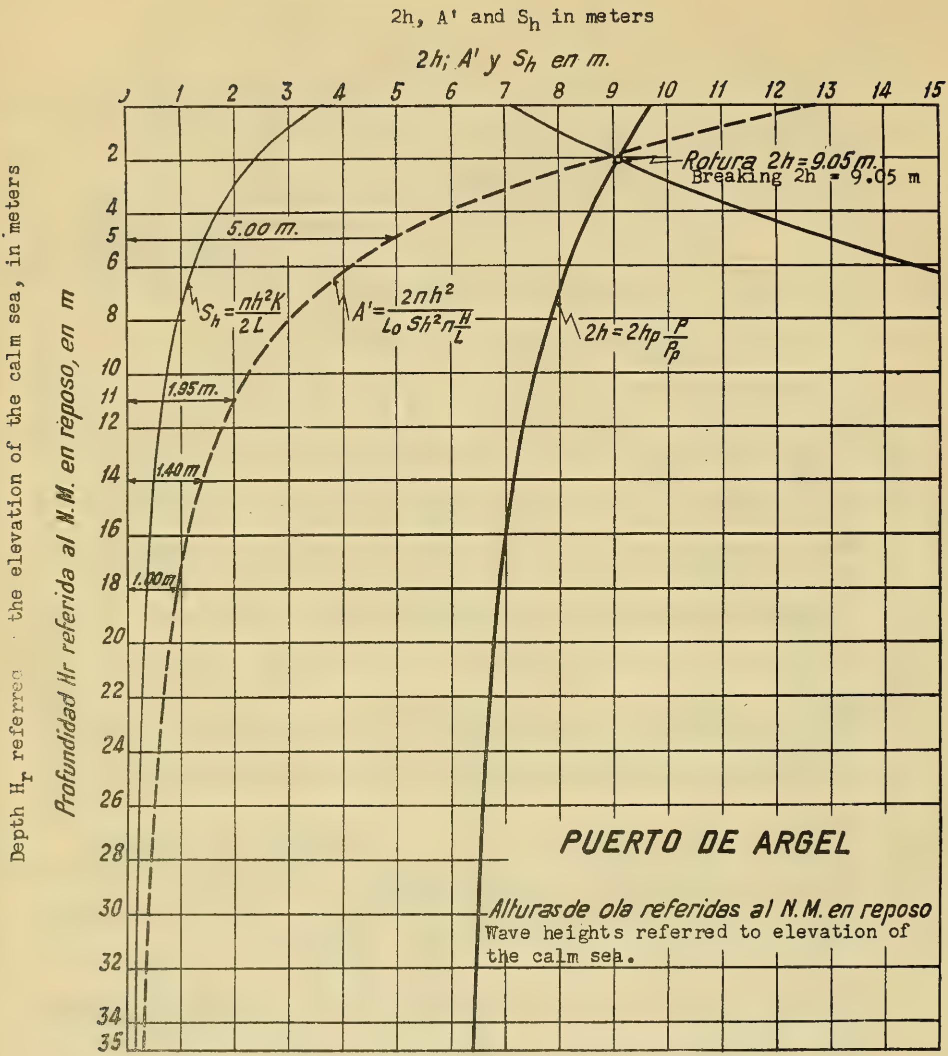

2h, A' and S_h in meters
2h; A' y S_h en m.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
Rotura teórica H=h
Technical Breaking H = h
Rotura: 2h=9.05 m.
Breaking: 2h = 9.7 meters
$S_h=\frac{nh^2K}{2L}$
$A'=\frac{2n\,h^2}{L_0\,Sh^2\pi\frac{H}{L}}$
$2h=2h_P\frac{P}{P_P}$
2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 35
Depth H referred to the mean elevation in movement, in meters
Profundidad H referida al N.M. en movimiento, en m.
PUERTO DE ARGEL
Increment of the height of wave over the batter of the dike.
Incremento de la altura de ola sobre el talud del dique
Altura de ola virtual en taludes sumergidos
Height of virtual wave on submerged batters.
$2h_p=6.45$ m.
Height of wave approaching the dike.....2 hp = 6.45 meters
Altura de ola que aborda el dique ______ $2h_p=6.45$ m.
Altura de ola al romper sobre el dique 2h = 9.05 m.
Height of wave on breaking over the dike... 2h = 9.05 meters

Figura 1'.



Height of wave approaching the dike ... 2hp = 6.45 m

Altura de ola que aborda el dique____2hp=6.45 m.

Altura de ola al romper sobre el dique_2h=9.05 m.

Height of wave on breaking over the dike... 2h = 9.05 m.

Figura 2.

$$\frac{\mathrm{ct}\,\alpha - 1}{\sqrt{1 + \mathrm{ct}^2\alpha}} = \frac{1.95}{1.7} \sqrt[3]{\frac{15 \times 2.7}{4000}} = 0.248$$

whence the batter should be $\mathrm{ct}\,\alpha = 1.43$

For that situated below $H_r = -14$ m:

$$\frac{\mathrm{ct}\,\alpha - 1}{\sqrt{1 + \mathrm{ct}^2\alpha}} = \frac{1.40}{1.7} \sqrt[3]{\frac{15 \times 2.7}{1000}} = 0.282$$

whence the batter should be $\mathrm{ct}\,\alpha = 1.51$.

For the deep zone $H_r \leq -18$ m, one obtains a weight:

$$P = \frac{15 \times 1.00^3 \times 2.7}{\left(\frac{1}{\sqrt{5}}\right)^3 \times 1.7^3} = 94 \text{ kilograms}$$

likewise quite acceptable, and the weight corresponding to the toe of the dike will be:

$$P = \frac{15 \times 0.33^3 \times 2.7}{\left(\frac{1}{\sqrt{5}}\right)^3 \times 1.7^3} = 3.4 \text{ kilograms.}$$

In this way we obtain a theoretical profile, shown in Figure 3' similar to 3, even more closely approximating the real profile; but the really interesting fact is that both figures, whose calculated wave heights differ by less than 8 per cent, a degree of approximation that we consider difficult for anything practical to really exxeed, are now authentically confirmed by the very important direct observation from this interesting compilation. It is also now confirmed undoubtedly, through authentic direct observation and despite the simplifications one is forced to introduce into the complex subjects of maritime engineering, that the degree of approximation really obtained is superior to that of many calculations of engineering on terrestrial subjects, in which, even legally there are imposed large safety factors, generally greater than two and frequently approximating three.

In the rock fill dikes the proportional increase of cost occasioned by these always convenient safety factors would not be greater than that for terrestrial works, and moreover, their cost would be recovered in no long time, on the basis that, by placing somewhat heavier stones or blocks the movements and consequent expenditures diminish and thus very costly outlays for maintenance are saved.

For these reasons it seems advisable, despite the very satisfactory confirmation just made and which one must remember refers to the strict equilibrium limit, to adopt at least a reduced factor of safety of 1.5 on the coefficients N, so that they

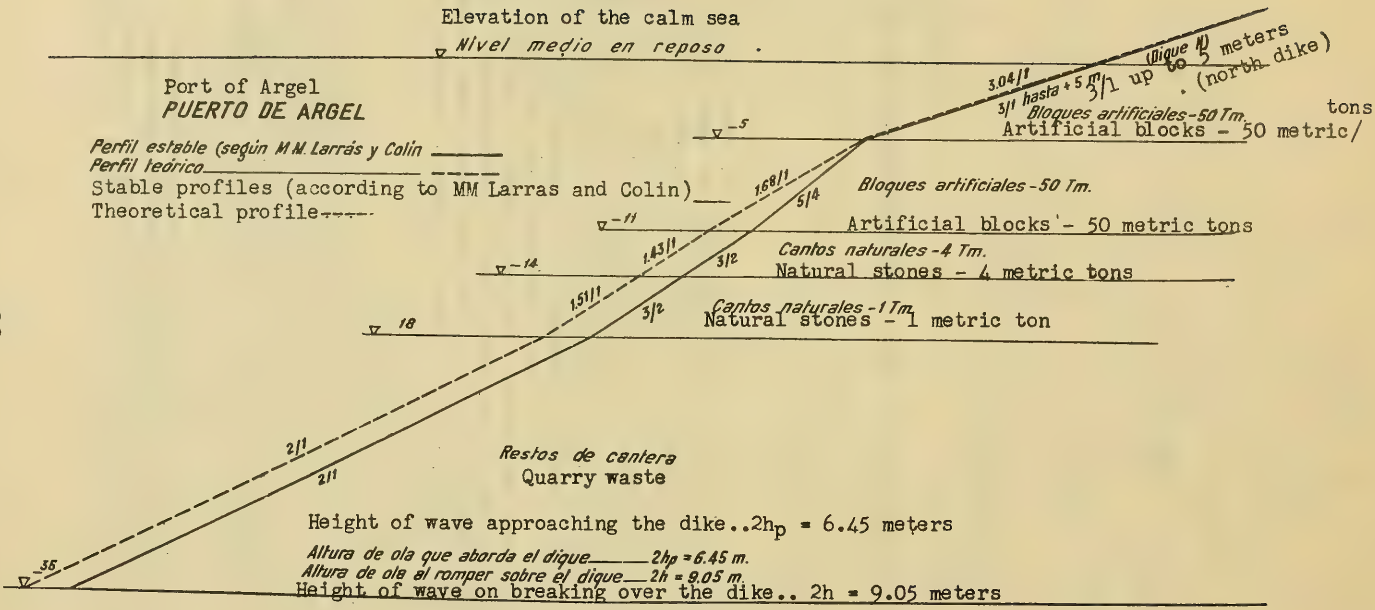

Elevation of the calm sea
Nivel medio en reposo
Port of Argel
PUERTO DE ARGEL
Perfil estable (según M.M. Larrás y Colin
Perfil teórico
Stable profiles (according to MM Larras and Colin)
Theoretical profile
(Dique N)
3/1 up to 5 meters (north dike)
3.04/1
3/1 hasta + 5 m
Bloques artificiales-50 Tm.
Artificial blocks - 50 metric/tons
-5
1.68/1
5/4
Bloques artificiales-50 Tm.
-11
Artificial blocks - 50 metric tons
1.43/1
3/2
Cantos naturales-4 Tm.
-14
Natural stones - 4 metric tons
1.51/1
3/2
Cantos naturales-1 Tm.
18
Natural stones - 1 metric ton
2/1
2/1
Restos de cantera
Quarry waste
Height of wave approaching the dike..2hp = 6.45 meters
Altura de ola que aborda el dique 2hp = 6.45 m.
Altura de ola al romper sobre el dique 2h = 9.05 m.
Height of wave on breaking over the dike.. 2h = 9.05 meters
35

Figura 3'.

would be raised to 29 and 23, respectively, for the artificial blocks and natural stones.

More, really, than the weight of these blocks or stones, this increase would affect the batters of the projects, flattening them a little, and it should be remembered that the said reduced safety factor of 1.5 would only suppose a margin of $\sqrt[3]{1.5} = 1.15$ in the wave height or in the size of stones.

In any event, and with respect for strict economy, one can neglect said reduced safety factor, continuing to use the primitive coefficients of 15 and 19 on large dikes where the stones are carefully placed to a considerable depth, provided that the increase of wave height produced by the work itself is always calculated following the indicated procedure.

On the other hand, and to simplify the calculations on the smaller dikes, said factor of 1.5 can be adopted, that is, coefficients N equal to 23 and 29, on the dikes whose stones are carefully placed to a lesser depth, and by virtue of this precaution being able to take the incident wave height rather than the height of wave that breaks over the dike, as already indicated textually in 1938.

The theoretical relative breaking depths, deduced from the cited table and the curve $\sqrt{K}\sqrt{a_0/a}$ of Figure 22 of the report, being $\frac{H}{L_o} \simeq 0.03$, to which a wave height amplification factor of 1.3 applies, it will be possible to calculate a rock fill dike on the basis of the incident wave height and adopting the cited coefficients 29 or 23 that include the safety factor 1.5, whenever the relative depth of its toe is less than $\frac{H}{L_o} = 0.06$, corresponding to the amplification factor of the cited curve $\frac{1.3}{\sqrt[3]{1.5}} = 1.13$, or whenever the depth of careful placing of the stone of the dike is less than $\frac{H}{L_o} \times L_o = 0.06 \times 250 = 15$ meters on our Cantabrian coasts, or $\frac{H}{L_o} \times L_o = 0.06 \times 150 = 9$ meters on the Mediterranean coasts.

As was made a matter of record in the XVII International Congress of Navigation convened in Lisbon, it is also highly satisfactory that the formula deduced in the American report of Epstein and Tyrrel for reflecting rock fill dikes, starting from our expression for pressures of reflection $P_r = \frac{CV}{g}$ *, is

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* See the cited report Calcultion of Vertical Dikes (Calculo de diques verticales).

similar to that which we obtained in 1938 for breakwater dikes.

In effect, the American formula is:

$$P_t = R_t \quad \frac{5H^3}{(S-1)^3\ (\mu - r)^3}$$

and ours:

$$P = \frac{NA^3 d}{(\cos\alpha - \sin\alpha)^3\ (d-1)^3}$$

in which

P_t = P = weight of the individual stones
H = A = 2h = wave height
s = d = relative density of the material
μ = natural batter of the rock fill $\simeq 1$
r = tan α = slope of the rock fill

R_t and N being the coefficients.

Expressing the American formula in our notation, it becomes:

$$P = R_t \cos^3\alpha \ \frac{A^3 d}{(\cos\alpha - \sin\alpha)^3\ (d-1)^3}$$

which is ours except only that it includes the factor $\cos^3\alpha$ in the coefficient.

It is a matter of record that on establishing our formula it was already indicated, that the coefficient should be able to vary with the data of the problem.

Practically the angle α , which varies most for the upper part of the dike, will not vary much, for from cotan $\alpha_1 \simeq 3$, corresponding to present rock fill dikes, it cannot get much steeper than cotan α_2 = 2, even in the reflecting dikes, because of the enormous weight of the stones this necessitates. Between those maximum limits the relation is:

$$\frac{\cos^3\alpha_1}{\cos^3\alpha_2} \cong 1.2$$

which would represent only a small difference in the weight of the stones and even less in their size, whose relation would be $\sqrt[3]{1.2}$ = 1.06. Only direct observation can determine properly N or R_t, including cases of very steep batters.

Concerning this last coefficient, it should be indicated that the expression for it determined in the American report can have reality only when the protecting blanket is made up of parallelopiped blocks perfectly aligned, with their three dimensions horizontal, normal to the slope, and following the line

of maximum slope, it being precisely under these theoretical conditions, of reduced joints, when the calculating procedure followed, based on internal pressures which must be transmitted through those joints, lacks applicability. If the blocks move through any cause or settlement, the cited three theoretical dimensions no longer really exist, the coefficient R_t becoming the N determined only on the basis of the overall dimension of the stone.

Although only under the heading of curiosity, it is also interesting to compare the results obtained for the theoretical value $A' = \frac{2\pi r^2}{LK}$ with the practical results pertaining to the limiting velocities of erosion on the bottom of a canal, whose bed we suppose horizontal, that is to say $\alpha = o$, and the formula that gives us the weight of the individual stones would be:

$$P = \frac{NA^3 d}{(d-1)^3}$$

as

$$A' ; \frac{2\pi r^2}{LK}$$

we get

$$P = \frac{8\pi^3 Nd}{(d-1)^3} \times \left(\frac{r^2}{LK}\right)^3$$

and since

$$v \max = \frac{\pi r}{T} \qquad \text{and} \qquad T = \sqrt{\frac{\pi LK}{g}}$$

one obtains

$$P = \frac{8 N d v_{max}^6}{g^3 (d-1)^3} .$$

Applying this formula for the velocity in the canal, $v \max =$ 1 meter/second, and with a mean specific weight of material that constitutes the individual stones or grains of d = 2.6 metric tons/ m^3, one obtains:

$$P = \frac{8 \times 15 \times 2.6 \times 1^6}{9.81^3 \times 1.6^3} = 0.081 \text{ kilogram},$$

which represents a cube of side:

$$l = \sqrt[3]{\frac{P}{1000 d}} = \sqrt[3]{\frac{0.081}{2600}} = 0.03 \text{ meter},$$

dimension of gravel whose order of magnitude is still in accord with observed reality for said limiting velocity of erosion, on horizontal bottoms, of 1 m/sec, in spite of the enormous extrapolation made, which it would not be permissible to carry on indefinitely. The formula gives acceptable results, without modifying the constants from those applying to stones or blocks of several tons weight, to these gravels that don't weigh 100 grams.

As a consequence of the foregoing, we estimate that we have confirmed authentically, through direct observations which always constitute the definitive ratification of these matters of technology, the generalization of the formula and the values of its coefficients, despite the simplifications whose introductions in these complex subjects are unavoidable.

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ANNOUNCEMENT OF PUBLICATION

The Beach Erosion Board announces the publication of its Technical Memorandum No. 21, "The Interpretation of Crossed Orthogonals in Wave Refraction Phenomena". Copies are being mailed to those individuals and institutions on the mailing list for technical publications.

A limited number of copies are available for distribution upon request to the President, Beach Erosion Board, Corps of Engineers, 5201 Little Falls Road, N. W., Washington 16, D. C.

APPLICATION OF ASPHALT IN HYDRAULIC ENGINEERING WORKS

by

J. H. van der Burgt

FOREWORD

The following translation was furnished the Beach Erosion Board by the author following a recent visit to the United States. It is published here to acquaint American engineers with Dutch progress in the use of asphalt mixtures for shore protection works. The paper has been abridged to permit publication in the Bulletin. The opinions and conclusions expressed are those of the author and not necessarily those of the Beach Erosion Board.

Introduction

The extensive application of synthetic asphalt in road construction work started around 1923, whereas it took another ten years before this material was applied to any extent in hydraulic engineering works. This is caused by the fact that the almost horizontal road surface encased by the shoulders, was ideal for testing the new and rather unstable material, whereas the sloping surfaces of hydraulic works were highly unsuitable for this purpose.

The share of the Netherlands in the research work regarding the application of asphalt in road construction and hydraulic works, has been appreciable. Extensive research done by the Rijkswegenbouwlaboratorium at Scheveningen and the Research Laboratory of the Royal Shell at Amsterdam, combined with thorough testing and observation during the execution of asphalt-works have led to such control of the properties of asphaltic bitumen and bituminous mixtures, that now it is possible to use this very durable product appropriately on sloping surfaces, even on steep slopes.

Asphaltic Bitumen and Bituminous Mixtures

The asphaltic bitumen is derived to a small extent from natural sources and synthetically made on a large scale.

The well known natural asphalt from Trinidad is a rather impure product, which is unfit for use in hydraulic works, as the enclosed clay-particles are affected by water.

The synthetic asphaltic bitumen is derived from petroleum. The remaining product becomes harder as more oily components are evaporated during the process of distillation.

*Composed after a lecture, given by the author on 10 Dec 1948 for guests and members of the V.B.W. (Association of Bituminous Works)

The asphaltic bitumen consists of tiny carbon particles approximately of equal size called micelles, which might be imagined as floating in an oily medium. The stability of the bitumen depends on the ratio of micelles and medium. Addition of a filler, i.e., a finely ground non-hygroscopic product increases the stability of the bitumen because, according to Professor Nellensteyn. the finest particles of the filler form new micelles, thus increasing the binding properties of the asphaltic bitumen.

Dependent on the kind of work, the asphaltic bitumen must meet certain standards, which are specified in the instructions for testing bituminous construction materials (K.V.B.B.). One of the most important points in the testing of asphaltic bitumen and asphalt mixtures for hydraulic works is the determination of the softness of the material, which is measured by the degree of softness or the penetration index.

Asphalt mastic or asphalt cement is the name of the mixture of asphaltic bitumen and filler.

Asphalt mortar or sand asphalt is the name of the mixture of asphalt mastic and sand.

Asphalt concrete is the name of the mixture of asphalt mortar with gravel or broken stone.

In using asphalt mixtures which are to be transported over a certain distance, one should take care to heat and preferably isolate the containers, whereas for liquid mixtures the solid materials should be kept in suspension by a special stirring device.

Standards for Asphaltic Bitumen and Asphalt Mixtures

For hydraulic works asphaltic bitumen and asphalt mixtures should meet the following standards:

1. Remain plastic, even after cooling down slightly;
2. Be sufficiently elastic, even at low temperatures, so that the material will follow if the underlying base settles unevenly;
3. Remain stable upon a slope, even above the water surface and at high temperatures;
4. Be proof against oxidation;
5. Stick to clean and dry surfaces;
6. Be proof against aggressive water (salt water, swamp water, etc.);
7. Be proof against abrasion by sand;
8. Be proof against wave action.

The combination of elasticity and stability mentioned under 2 and 3 can hardly be expected in any material, but it has been proven that a satisfactory compromise can be achieved.

Of course it depends entirely on the kind of work for which the material is to be used, which of the above mentioned properties should be predominant or whether any other standards should be met.

General Review on the Application of Asphalt Mixtures

For hydraulic works the asphaltic bitumen is mostly used in a mixture, in which the percentage of the expensive asphaltic bitumen is kept as low as possible. The other materials such as the filler (partly), sand, gravel and broken stone are natural products, the gradation of which greatly influences the quality of the mixture. Therefore, to efficiently obtain reliable results, laboratory control and information are of great value.

In working with asphalt mixtures, one should keep the temperature as high as possible until the job is reached. Care should be taken that no superheating occurs, since then the asphaltic bitumen will be cracked, which means that free carbon is formed, changing the mechanical properties unfavorably.

In many instances where hydraulic works are constructed the water can be retained temporarily so that the asphalt mixture can be applied on a dry base. In other areas subject to tidal- and wave-action it often occurs that the job must be completed within a short period of time and the mixture has to be applied to a wet base or even under water. In the latter case one cannot depend on adhesion and special measures must be taken in order to carry the asphalt mixture into place at the required temperature. The recent experiments in the 1st kerosene-harbour at Rotterdam, where a plastic mixture was carried to its destination through isolated tubes, indicated that it is quite possible that this material can even be applied under water. Certainly, the last word regarding these problems has not been said yet, but by proper cooperation between the research institutes and those who design and execute hydraulic works where bitumen is being used, the solution of these problems can definitely be found.

Cold and Hot Asphalt

Asphaltic bitumen is used in hydraulic works as cold asphalt and as hot asphalt.

Hot asphalt may be used in:

1. Permeable works.

   a. by penetration with pure asphaltic bitumen
   b. by penetration with asphalt mastic
   c. by sheeting with bituminous sand

2. Impermeable works.

   a. by surface-treatment, with pure asphaltic bitumen
   b. by sheeting with asphalt mortar (sand asphalt)
   c. by sheeting with asphalt concrete
   d. by grouting with pouring asphalt.

Cold Asphalt

This material is liquid at normal temperatures as a result of the addition of certain volatile components which evaporate and thus cause the mixture to stiffen gradually. Therefore, as a rule this material cannot be used under water or in thick layers as it remains soft. It would be worth while to improve the quality of this product, because it may prove of great value where hot asphalt cannot be used or where repairs must be carried out. Cold asphalt was used in 1936 at a trial section on the outer slope of the north east polder dike near Urk.

Cold asphalt grouting containing 16% of asphalt emulsion, 25% of cement, 53% of sand and 6% of water was applied at normal Amsterdam level (N.A.P.). On each square meter of the basalt stone slope protection 19 kg of grouting material containing 3 kg of asphalt emulsion were used. This grouting was often submerged and almost constantly subject to wave action and has gradually vanished.

Hot Asphalt

This product has been used in various compositions both below and above the water surface.

To apply thin layers (surface treatment with pure asphaltic bitumen or asphalt mastic) above water, a dry and dust free base is essential for good adhesion. For thick layers (sheeting with bituminous sand, asphalt mortar or asphalt concrete) a stable mixture is required, which often should be elastic as well.

On a wet base or under water there will never be any adhesion between the base and the bituminous material and therefore the mixture should be carried to its destination in such a way that the internal heat is preserved as long as possible. This can be achieved by applying the material in solid masses of substantial size, for instance by the aid of clamshells, shoots or isolated tubes. Thus, even under water the masses will stick together where they touch, because the surfaces which cooled will be reheated by the radiation from the centre. Thanks to this phenomenon asphalt grouting may penetrate deeply (1 to 1.5 m under water). This is necessary, not only to envelop the stone in order to anchor it safely to its base, but also to fill the cavities. In works along the seashore this is imperative in order to insure that the wave and groundwater action does not result in internal water and air pressures which may have a disastrous effect. If the voids between the stones are too small, the bitumen or the mixture cools off too rapidly and there will be little penetration, resulting in unsound work. Therefore, the kind of works and the forces to be resisted determine to what extent the cavities should be filled. Especially in sea-work one should be on the alert against being penny-wise and pound-foolish.

If the asphalt mixture travels freely through the water over a certain distance or is exposed to wave action while it is being poured, the mixture will change into a porous mass which hardly penetrates at all, with unreliable work as a result. It has been found that in poor mixture which are submerged permanently or regularly, the film of asphaltic bitumen which covers the particles is gradually replaced by water, causing the mixture to lose its coherence and fall apart. Since it is impossible to coat the wet layer of porous material with an impermeable layer of asphaltic bitumen or tar, it is recommended that a sufficiently rich mixture be chosen initially for layers which are constantly or regularly submerged by water.

In the following description of works executed with the use of hot asphalt, the above-mentioned points will further be discussed where necessary.

Works Executed with the Use of Hot Asphalt

Since hot asphalt has been used in a great many hydraulic works, it is impossible to give a complete record in this short review. Therefore only a description of a number of applications will be given for the following constructions.

1. bank protection along canals
2. impermeable sheeting of the wetted profile of canals
3. impermeable sheeting of weirs
4. bank protection along rivers
5. slope protection along sea shores
6. mattresses
7. reinforcement of beach groins and harbour-dams.

The original translation contained discussions of the first four types of works; these have been deleted in this presentation.

Slope Protection Along the Sea-shore

The following applications of asphaltic bitumen or asphalt mixtures have been made.

Trial Section on the Outer Slope of the "Northeast Polder" Dike Near Urk 1936

At this trial section (total area of 2000 m^2) tests were made in cooperation with and under control of the Research Laboratory of the Royal Shell at Amsterdam. The slope of the dike ranged from 1:4 at the bottom to 1:3 at the top, separated by a 5 m terrace at about 2.00 m + N.A.P. (Normal Amsterdam Level).

Test 1a. Penetration of macadam with pure asphaltic bitumen. The area extends from +2.00 to +3.50 m (Normal Amsterdam Level) and is covered respectively by a layer of set bricks, an 8 cm layer of dumped rubble stone and an 8 cm layer of compacted broken stones of 3-5 cm. This last layer was penetrated with 9 kg/m^2

asphaltic bitumen 60/70 covered with a 1, 5 cm layer of broken stones of 1-2 cm which was treated in turn with a surface treatment of 3 kg/m^2 asphaltic bitumen 60/70 and finally protected by a 1.5 cm compacted cover of broken stones of 1-2 cm. This protection has served well although attacked heavily during southwest gales.

Test 1b. Penetration of macadam with asphalt mastic. The area has approximately the same cover as described under 1. It reaches from N.A.P. to 3.50 m + N.A.P.. The lower part is subject to wave action daily. Bulges have formed in this lower part, the cause of which is unknown. Possibly sand was deposited between the rubble stones during the construction. Water may have accumulated in this sand creating a pressure behind the impermeable cover. Since the voids in the rubble stone are rather small anyway, it is imperative that care be taken to prevent sand and fine material filling the voids.

It seems undesirable to use the procedures described under 1 and 2 under less favorable conditions although they did prove satisfactory under the circumstances of the tests.

Test 2d. Grouting of set stone with pouring asphalt. Three areas were covered, reaching from N.A.P. to +0.90 and +2.00 m N.A.P. respectively. The stones were set in the usual way, after which the spaces were filled with rubble stone, gravel and sand to 8 cm from the surface. Then the sides of the stones were treated with a cold bituminous priming coat, after which the spaces were filled with pouring asphalt to 2 cm from the surface. The pouring asphalt consisted of 43, 3% DX 10, 4.7% asbestos fiber 2-4 mm and 52% sand 0-2 mm. Twenty-two kg per square meter of pouring asphalt were used, containing 9.5 kg of asphaltic bitumen. The lower parts of the spaces were kept open to let the water pass freely and thus prevent the building up of pressure under the layer.

This construction appears to be very solid, has served well, and seems sound and economical at exposed slopes.

Protection of the Dune Foot of the "Green Beach" at Terschelling 1937 (Figure 1)

In 1937 the beach near the dune foot at one end of the "Green Beach" at Terschelling was repaired and protected after it had been damaged by the sea during high tides. The beach was

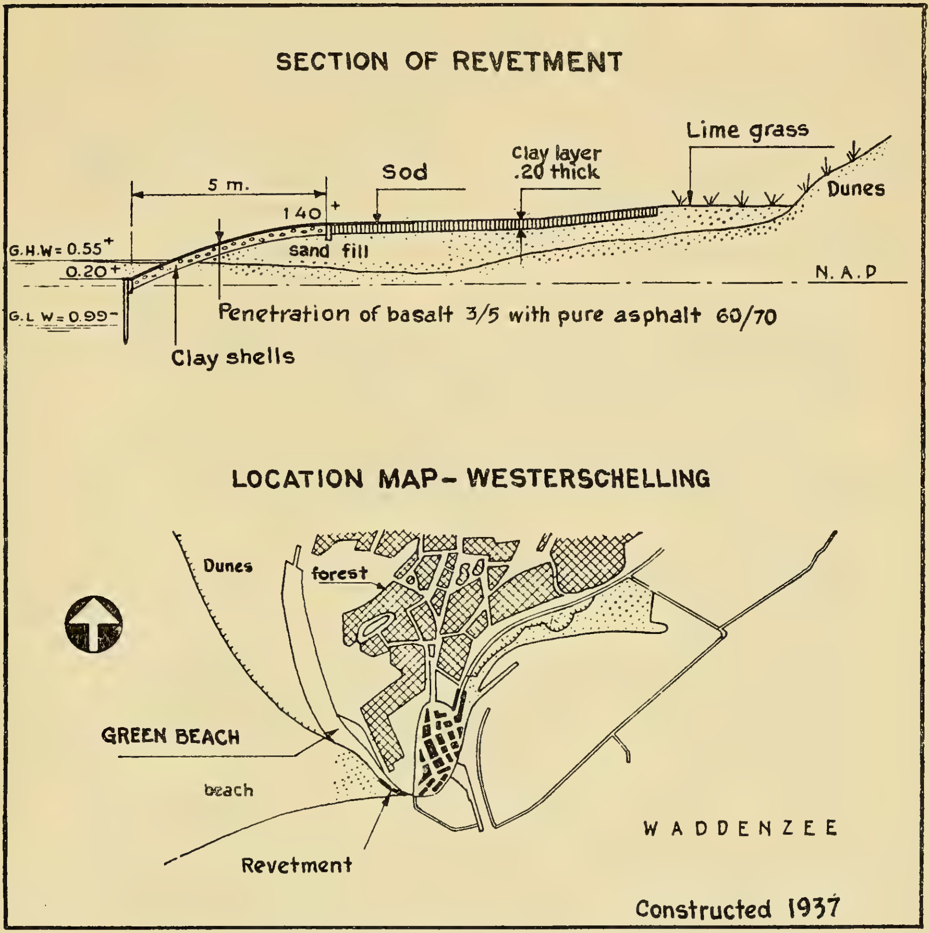


Figure 1. Protection of the "Green Beach" at Terschelling.

covered with a 20 cm layer of so-called shell-clay, which is a natural deposit often found in the Wadden-sea, containing clay and shells and being slightly permeable. Three layers of compacted broken basalt stone (3-5 cm) were placed upon this, each of which was penetrated with pure asphaltic bitumen 60/70. Although the cambered slope was rather flat, the construction proved unstable and slides occurred. Probably high ground water pressure under the construction was the main reason for this failure since a high range of dunes was near. The bituminous protection was removed after some years and replaced by a brick pavement on shell-clay.

Repairs On the Outer Slope of the Westkapel Seadike, 1946-47, (Figure 2)

The Westkapel sea dike was badly damaged during the struggle for Walcheren by bomb and shell explosions over a section north of the gap which was made in 1944. This damage was spreading gradually under the action of high tides.

Part of the slope protection was repaired with concrete; elsewhere pouring asphalt was used extensively to form a protection in a quick and efficient way out of the loose stones which were present on the slope.

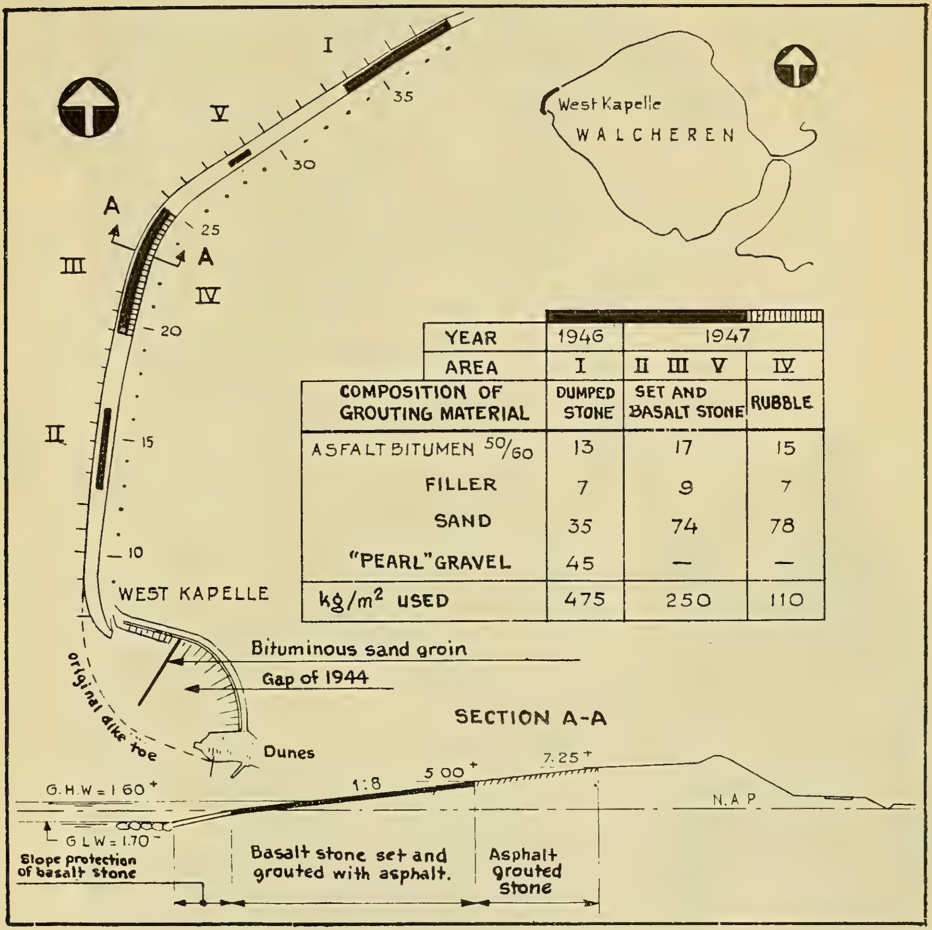
Furthermore, part of the slope was protected at the top by a layer of rubble stone penetrated with asphalt.

Test 1b. Penetration of rubble stone with asphalt mastic. Area IV, from +5.00 to +7.25 m N.A.P. was protected by a 0.20 m layer of dumped rubble stone, penetrated with asphalt mastic (15% bitumen, 7% filling and 78% sand), in such a way that an impervious asphalt cover was obtained at the surface.

The rubble stone layer was filled to a depth of 10 cm by 110 kg of asphalt mixture per m^2 (Figures 3 and 4). In order to prevent the spaces in the rubble stone being filled with water which might create a dangerous pressure, the penetrated section was sealed at the top by an asphalt coffer. This construction has proved satisfactory, but it cannot resist water pressure from beneath.

Test 2d. Grouting of set stone and dumped stone with pouring asphalt.

In 1946 an area was repaired by simply arranging the old and worn basalt stones in a more or less orderly manner and filling the voids with pouring asphalt. As the voids were large (50%) an average of



YEAR	1946	1947	
AREA	I	II III V	IV
COMPOSITION OF GROUTING MATERIAL	DUMPED STONE	SET AND BASALT STONE	RUBBLE
ASFALT BITUMEN 50/60	13	17	15
FILLER	7	9	7
SAND	35	74	78
"PEARL" GRAVEL	45	—	—
kg/m^2 USED	475	250	110

Figure 2. Reinforcement of the outer slope protection of the Westkapel sea dike, partly with asphalt mixtures.

Figure 3. Penetration of the rubble stone protection on the outer slope of the Westkapel sea dike from 5.00 to 7.50 m above NAP.

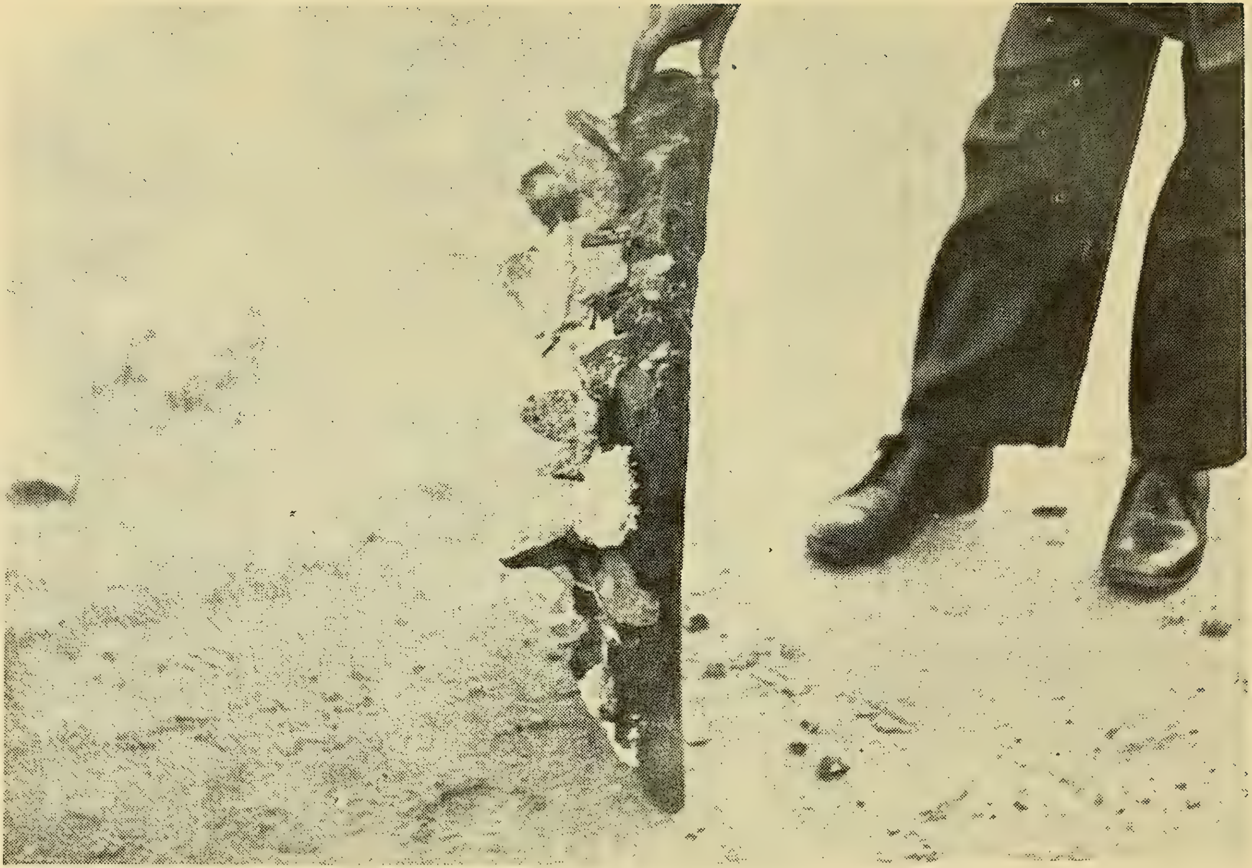


Figure 4. Westkapel sea dike. A fragment torn out of the rubble stone protection penetrated with asphalt mastic.

475 kg/m^2 of pouring asphalt, consisting of 13% asphaltic bitumen 50/60, 7% filler, 35% dune sand and 45% "pearl" gravel was used. In 1947 other areas were repaired by roughly setting the basalt stones on a thin base of rubble stone and filling the spaces with pouring asphalt, consisting of 17% asphaltic bitumen 50/60, 9% filler and 74% dune sand. 250 kg per square meter of pouring asphalt were used, which filled the spaces in the rubble stone base as well. About 12 kg/m^2 of asphalt remained on the upper ends of the stones. This was removed by wave action (Figures 5, 6, 7, and 8).

The protection built in this way proved satisfactory; practically no damage was caused by gales. The designers state that this construction method must be considered as an experiment. Due to the urgency of the work and the lack of time, there was no opportunity to investigate the economy and efficiency of this construction in comparison with others which might have been preferable. In spite of this the work, which was executed within a short period of time, may be considered as a success.

Sheeting of the New North Harbour Dam at Harlingen 1947-48, (Figure 9)

Quite a different kind of experiment with the application of asphaltic bitumen on a large scale is found in the protection of the new North harbour dam at Harlingen, where a dam consisting of sand was simply covered with a layer of so-called bituminous sand, that is a warm mixture of 5% asphaltic bitumen and 95% sea sand. This material was produced in the Netherlands for the first time by the Research Laboratory of Royal Shell at Amsterdam in the spring of 1947 at the road building section of the Dutch Concrete Association Ltd. (Hollandsche Beton Maatschappy) at Amsterdam. A small addition of asphaltic bitumen proved sufficient to bind the sand-particles strongly. The 160 m test section of the North harbour dam at Harlingen, which was built in the same year, was protected to the low water level by a layer of bituminous sand containing 5% asphaltic bitumen and 95% sea sand, 0.40 m thick to +4.00 m N.A.P. and 0.25 m thick above that level.

The under water slope protection on the sea side, which was constructed as a bituminous mattress, crumbled after it had been undermined as it did not have the required flexibility. Therefore during 1948 the existing work was re-covered over about 100 m length with a layer of a richer mixture containing 16% asphaltic bitumen (40/50) and 84% sea sand.

In 1948 the 900 m harbour dam was completed (figures 10, 11 and 12). The 1947 method was used once more with the exception that:

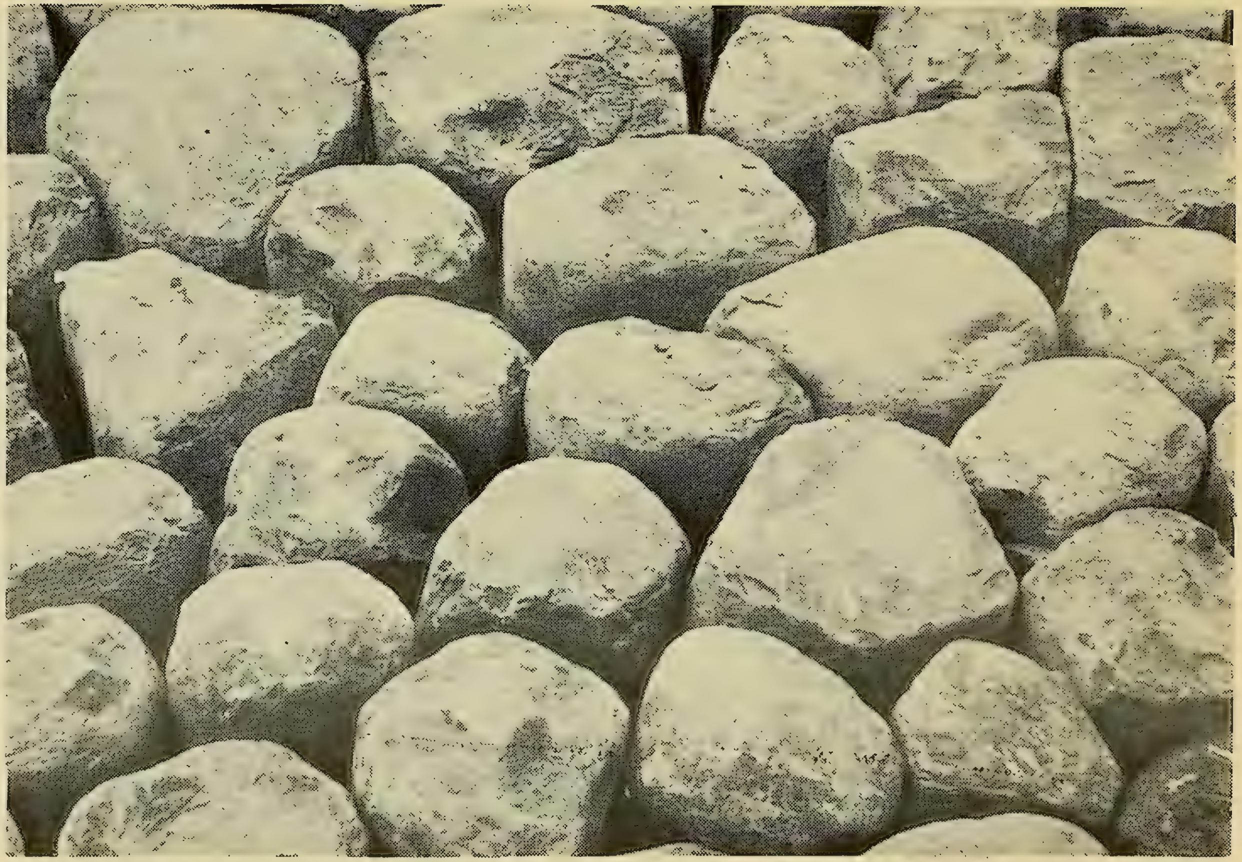


Figure 5. Westkapel sea dike. Riprap of worn basalt before the application of the pouring asphalt.

Figure 6. Westkapel sea dike. Westkapel sea dike. Riprap of worn basalt after application.

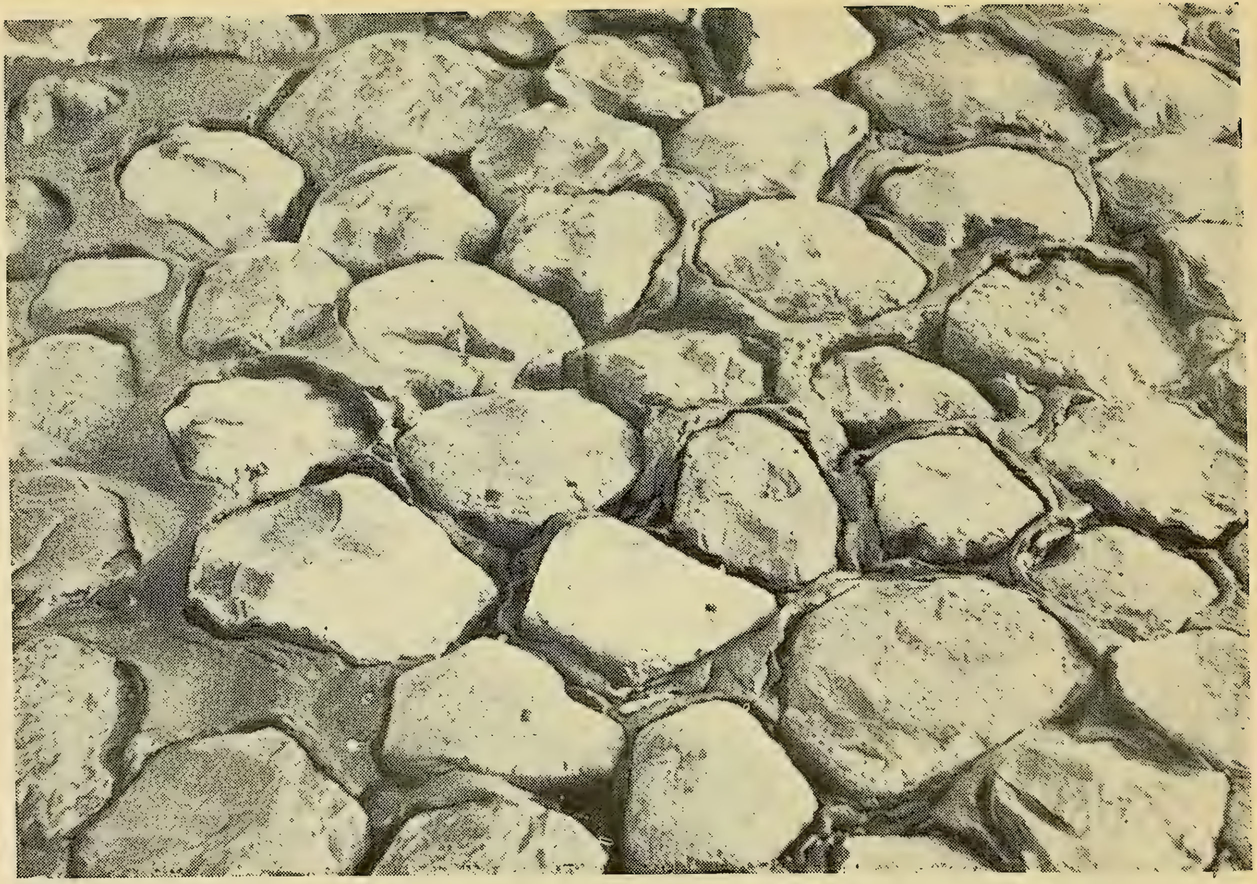


Figure 7. Westkapel sea dike. Worn basalt stones, roughly set, four months after the application of asphalt. The asphalt mortar which did not penetrate into the voids has been removed. by wave action.

Figure 8. Westkapel sea dike. Slope protection of basalt stone grouted with pouring asphalt.

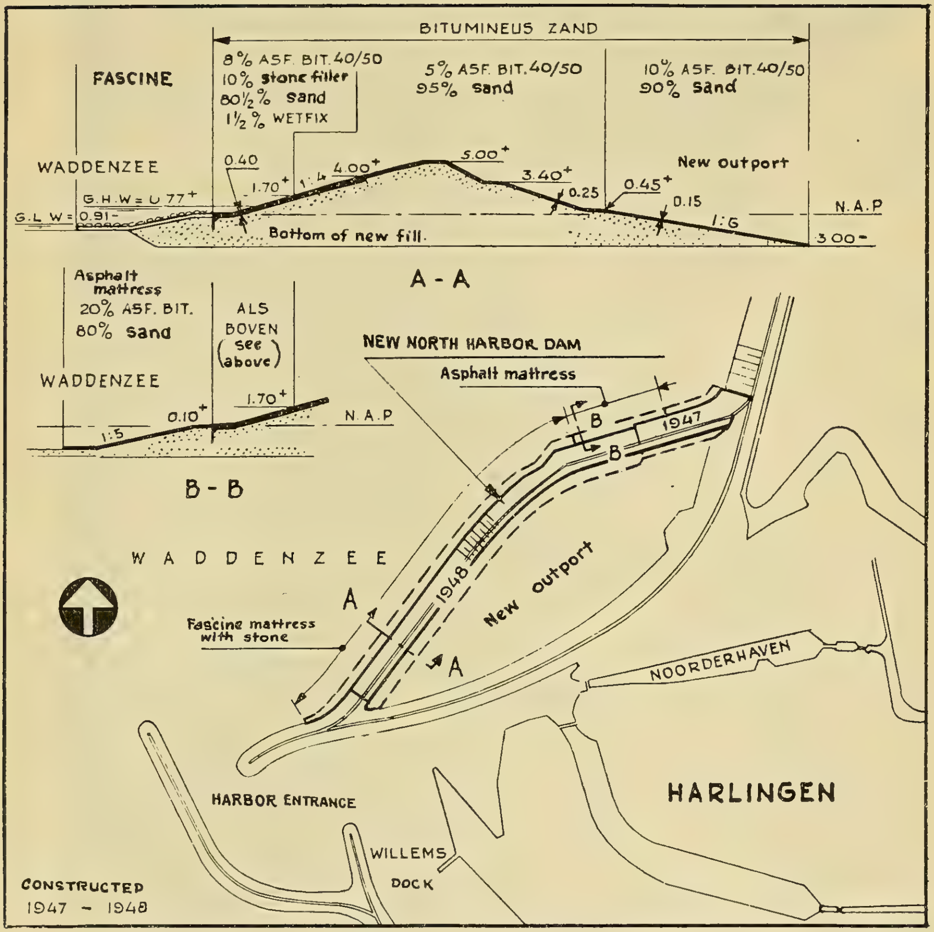


Figure 9. Sheeting of the new north harbor dam at Harlingen with bituminous sand and asphalt mattress.

Figure 10. New North harbor dam at Harlingen. Dumping and spreading bituminous sand on the outer slope.

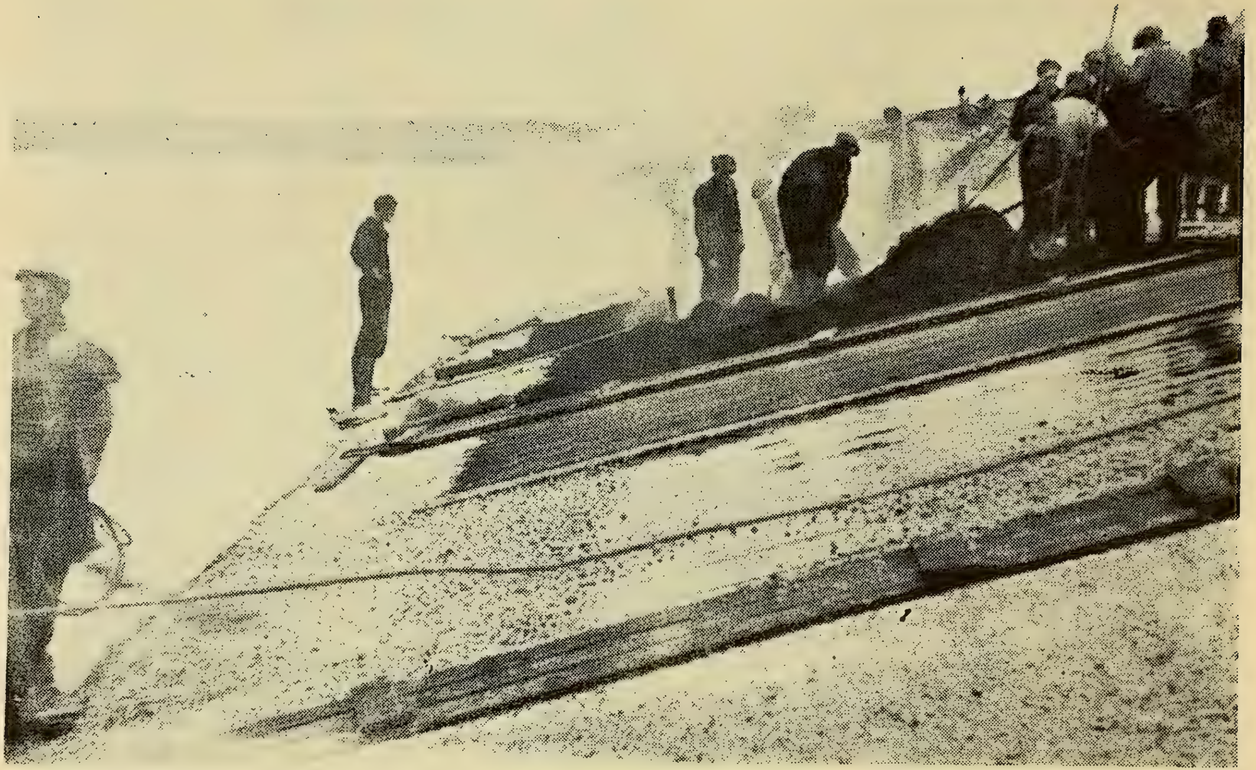


Figure 11. The cover of the outer slope of Harlingen dam being spread, compacted and covered with hot asphaltic bitumen and shells.

Figure 12. The closing gap of the harbor dam at Harlingen. In the foreground is a section of the initial dam at the foot of the new dam, which is protected with bituminous sand. In the rear, the section constructed in 1947.

1. the under water slope protection at the side of the Wadden sea was made of fascine mattresses, covered with stone.

2. the terrace at +0.10 m N.A.P. and the lower part of the outer slope were covered with a rich mixture, containing 8% asphaltic bitumen (40/50), 10% filler, 80.5% sea sand and 1.5% Wetfix.

3. the under water slope protection on the harbour side below the terrace at +0.45 m N.A.P. was laid on a dry base and consisted of bituminous sand containing 10% asphaltic bitumen and 90% sea sand

4. the remaining surfaces were covered with bituminous sand containing 5% asphaltic bitumen and 95% sea sand, sheeted with an impermeable layer of pure asphaltic bitumen or tar, and covered with shells.

Modification 1 was necessary because it had been proven that the bituminous sand could not follow the local changes of the sea bottom due to wave action and strong currents. On a short section of the under water slope prefabricated asphalt mattresses were placed, containing about 20% asphaltic bitumen and 80% sea sand. These will be discussed later in the section on mattresses.

Modification 2 proved necessary because in the original mixture (5% asphaltic bitumen and 95% sea sand) the adhesion diminished as a result of the replacement of the asphalt film around the particles by water. It was impossible to provide this area, which remained wet constantly, with an impermeable sheeting.

For the same reason the under water slope on the harbour side where no substantial changes of the bottom were to be expected, was provided with a rich cover of bituminous sand.

The impermeable sheeting mentioned under 4 seemed desirable to check the weathering of the porous layer which would be exposed to waves running up the slope and to rain water, and furthermore to form an acceptable construction from an aesthetic point of view.

Post war conditions, which hampered the import of basalt from Germany and the lack of foreign currency, were the main reasons for this large scale experiment. The construction withstood the high tide of 1 March 1949 in an excellent manner and otherwise lived up to expectations. The poor mixture however remains rather soft; and it remains to be seen whether the material will last.

This experiment is very important to determine the solidity of poor asphalt mixtures of various compositions as a temporary

or permanent slope protection along the sea shore.

Mattresses

Along the lower part of the Mississippi where the strong current undermines the banks along the bends, mattresses of asphalt mortar have been used extensively as a bank protection since 1932.

In April 1934 a great floating asphalt plant was put into use for this purpose. With this plant mattresses could be made 65 m in width and 190 m in length, reaching down to 50 m below low water and covering slopes of 1:3 to 1:5. The maximum current velocity was 2.4 m/sec.

In one season 7500 m^2 of asphalt mattresses, 5 cm thick were placed. The composition of the asphalt mortar was 12% asphaltic bitumen 30/40, 22% filler (loess) and 66% river sand. The mattresses were reinforced with steel wire netting (5 x 10 cm) and provided with steel wire ropes, spaced at 0.9 m, for lowering them into place.

In 1935 and 1936, reinforced asphalt slabls of 7 x 16.5 m were used on the beach groins near Galveston and Florida City. These slabs were placed under water with the aid of floating cranes.

This method was also used on mattresses of asphalt mortar, size 13 x 5 m and 0.15 m thick, weighing 20 tons, which were used on the outer slope of the North harbour dam at Harlingen (Figures 13, 14, and 15). These asphalt slabs contained 20% asphaltic bitumen and 80% sea sand, and were reinforced with four steel wires of 1" diameter. Some slabs cracked while they were being lifted, probably due to insufficient stretching of the wires before and during the fabrication (Figure 16).

Other mattresses of asphalt mortar, 15 x 5 m and 0.05 m thick were placed nearby. These were fabricated at Harlingen and wrapped around a drum 2 m in diameter after which they were transported and put into place (Figure 17). Both types of mattresses, which emerge during low water, have fitted themselves perfectly to the uneven base and show few if any defects.

Three slabs wrapped around drums were placed on the dumped stone in front of the north east polder dike. The upper part of these slabs has been destroyed completely; they contained only 15% asphaltic bitumen and proved to be very porous, probably as a result of insufficient stirring of the mixture during transportation from the point to the construction site.

In preparation for the proposed drainage of the Yssel lake several asphalt mortar slabs 2 x 2 m, 0.06 m thick, and of varying composition and reinforcement, were fabricated in 1948. The best result was obtained with a mixture of 15% asphaltic bitumen, 15%

Figure 13. New north harbor at Harlingen. The casting of an asphalt mattress (13 x 5 m; thickness, .15m) being completed on top of first layer (.075 m thick) on which the lifting cables were placed and strung.

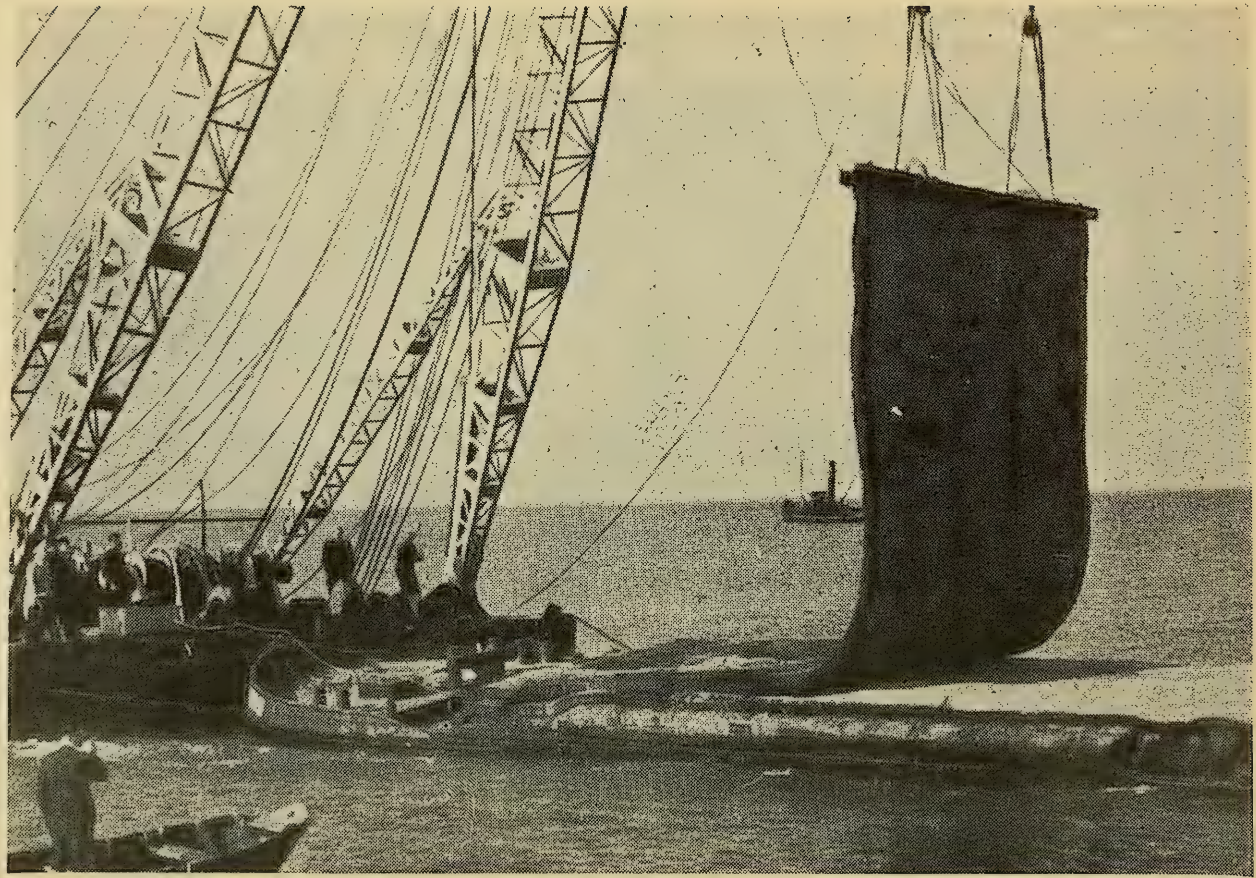


Figure 14. Lifting the 20 ton asphalt mattress shown in Figure 13 with a floating crane.

Figure 15. Placing the 20 ton asphalt mattress.

Figure 16. Twenty-ton asphalt mattress breaking while being lifted by crane.

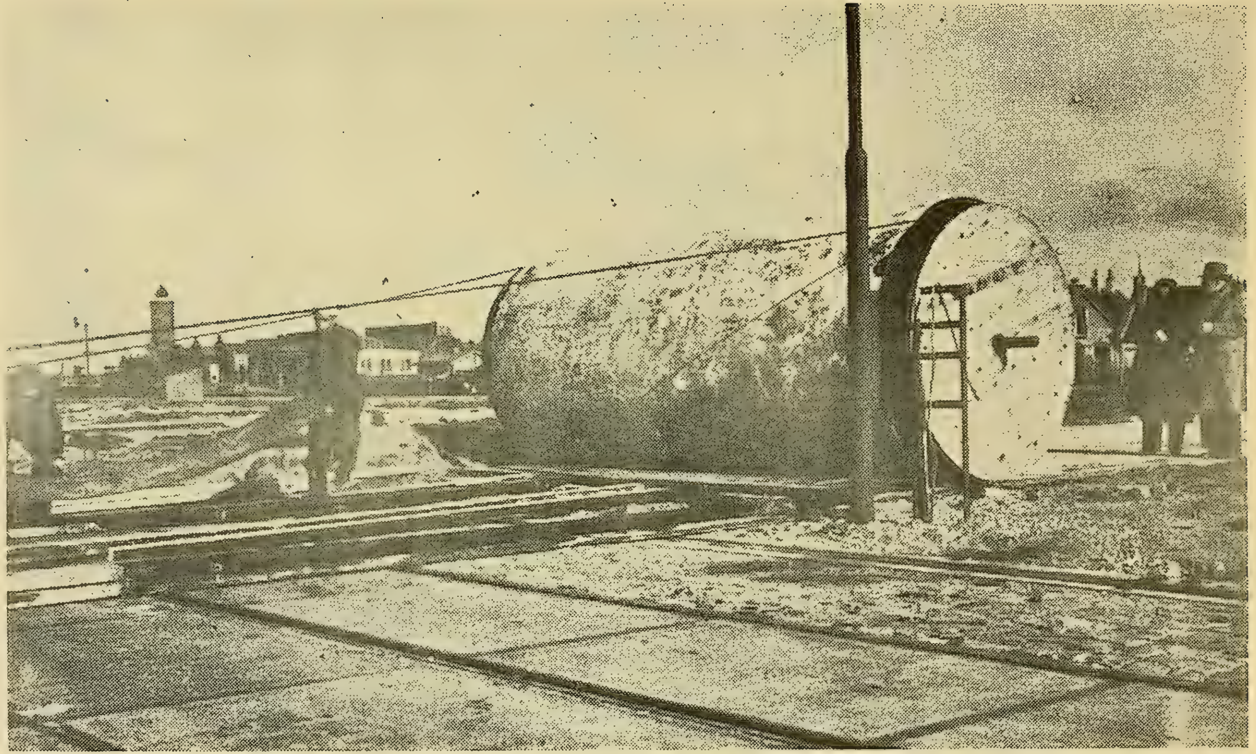


Figure 17. An asphalt mattress, 15 x 5 m, .06 m thick, weighing 9 tons, being wrapped around a drum. (Harlingen).

filler and 70% Yssel lake sand, and reinforcement of cocos matting with 0.015 m gauge sprayed with a binder before use.

The methods described above for construction, at the site or elsewhere, transportation and placing of asphalt mattresses can only be used in quiet water, whereas the slabs must be of restricted size when floating cranes or drums are being used. For some time therefore attempts have been made to find a method to transport and sink asphalt-mattresses, similar to the one which can be used with fascine mattresses. Also, if an asphalt mattress of any size could be kept afloat with inexpensive means and could be towed through a medium swell, the skilled fascine mattress workers could be used to assist in the difficult handling and placing of the mattresses, which would be of great value. In the system of W. J. van der Oord this can be achieved by providing the asphalt mattress with a temporary flexible rim of some impermeable material, thus forming a vessel of sufficient buoyancy.

A mattress of asphalt mortar was constructed for the piers at Hook of Holland according to this system. The mixture contained 20% asphaltic bitumen 60/70, 10% filler and 70% dune sand. The slab, 7.5 x 3 x 0.07 m, was reinforced with cocos matting of 0.015 m gauge and three towing cables of wire rope. The slab was constructed on a horizontal wooden floor which was built at a small beach along the New Waterway near Hook of Holland, at 0.80 m below the high water level (Figure 18). The slab floated as soon as the water reached 0.20 m above the floor and was towed out on the river. The asphalt slab and rim followed the wave movement easily (Figure 19). Presently the slab was towed back and anchored above the floor after which water was admitted through the tube in the centre and the slab was sunk for the time being to await transport to its final destination.

Although dependent on the most suitable construction method, it is to be expected that the cost of such a mattress will be considerably less than the cost of a fascine mattress with dumped stone.

The advantage of an asphalt mattress over a fascine mattress is that in sea water the former will not be attacked by the ship worm (teredo navalis). Experience will show whether the asphalt mattress will stand up under current and wave action and whether it is suitable for the protection of steep and uneven submarine slopes.

Reinforcement of Beach Groins and Harbour Dams

One of the great difficulties met in the construction of beach groins and harbour dams, is the removal of dumped stone and set stone by wave action. This is true particularly near deep water, where the effect of a heavy ground swell may be

Figure 18. Piers at Hook of Holland. Asphalt mattress 7.5 x 3 x .07 m immediately after casting. The casting floor is situated at .10 m + NAP = .80 m below mean high water level.

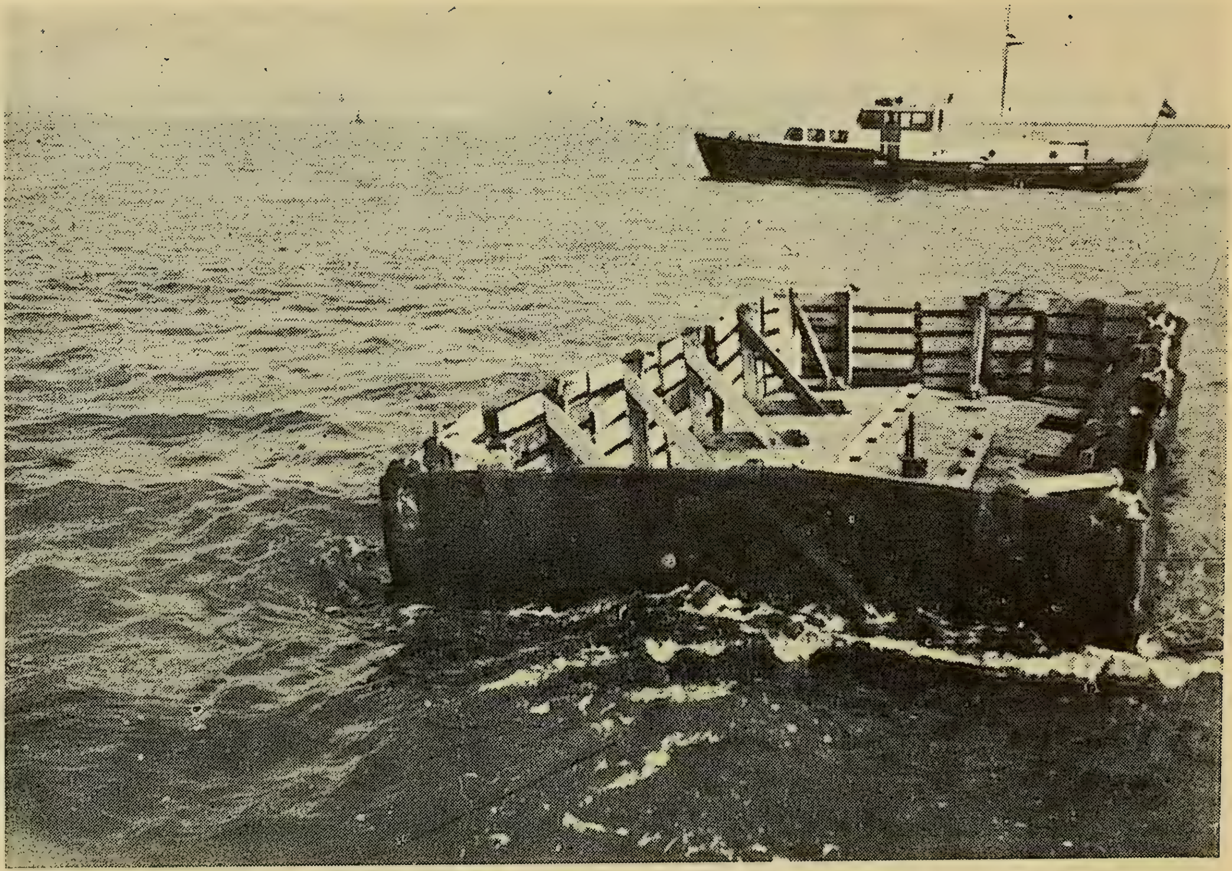


Figure 19. Piers at Hook of Holland. An asphalt mattress (7.5 x 3 x .07 m, draft .2m) being towed across the new waterway. The temporary rim of this mattress is made of impregnated canvas held up by loose wooden supports. In the center is a tube for the admission of water.

considerable, and rocks up to 3 tons and more have been moved.

In the United States an attempt was made to consolidate the stones, for the first time in 1936 by filling the voids with asphalt mortar. This method was used in the Netherlands for the first time in 1938 by the Engineer in Chief of Delfland, A. C. Kolff, on some beach groins near Scheveningen. Since 1945 it has been increasingly used.

a. Groin at the mouth of Columbia river 1936. - The groin is situated at the Pacific Ocean in an area of heavy wave-action and reaches from 18 m below to 7.5 m above low water. It has a top width of 12 m and is constructed of stones from 1 to 12 tons. The asphalt grouting contained 15-18% asphalt mortar and was applied only to the upper part.

b. Galveston Jetty 1936. - The asphalt-grouting of this structure proved satisfactory.

c. Delfland Groins. - The voids of the groins were filled with an asphalt mortar containing 20% asphaltic bitumen [50/60 (6%), 60/80 (7%) and 80/100 (7%)], 10% filler and 70% dune sand. Three groins were treated from 1938 to 1940 and seventeen more from 1946 to 1948. On each groin 300 tons of the mortar were used, or an average of 0.5 tons per m^2. This reinforcing method of the beach groin proved entirely satisfactory. When a floating mine exploded against one of these groins it was found that the voids were filled to a depth of 1 m below the surface.

d. South Jetty of the outer shipping channel at Ymuiden, 1938. - This jetty which was often badly damaged during heavy gales, was provided with asphalt mortar as described under c. Between the dumped stones 0.8 tons of asphalt mortar per m^2 and between the set stones 0.5 tons per m^2 were used. Since then no more damage has occurred.

e. North- and South piers at Hook of Holland 1946-50, (Figure 20). - The piers are 2 km in length, with the top, 4 m wide at mean sea level. The heavy stone protection (stones of 0.1 to 3 tons) of the outer section could not resist the heavy ground swell so that it was often damaged, and the normal gauge railroad which was laid on oak girders on top of the jetty required abnormally high expenditures for maintenance. In view of this, it has been decided to convert the railroad into a single lane road for trucks and to grout the set stone and dumped stone with asphalt-mortar where the attack is heaviest.

The outer 1 km section of the North pier and the so-called connecting jetty, forming the connection between the South jetty and the low jetty which was constructed at a later date, were thus reconstructed (Figure 21-26) from 1946 to the middle

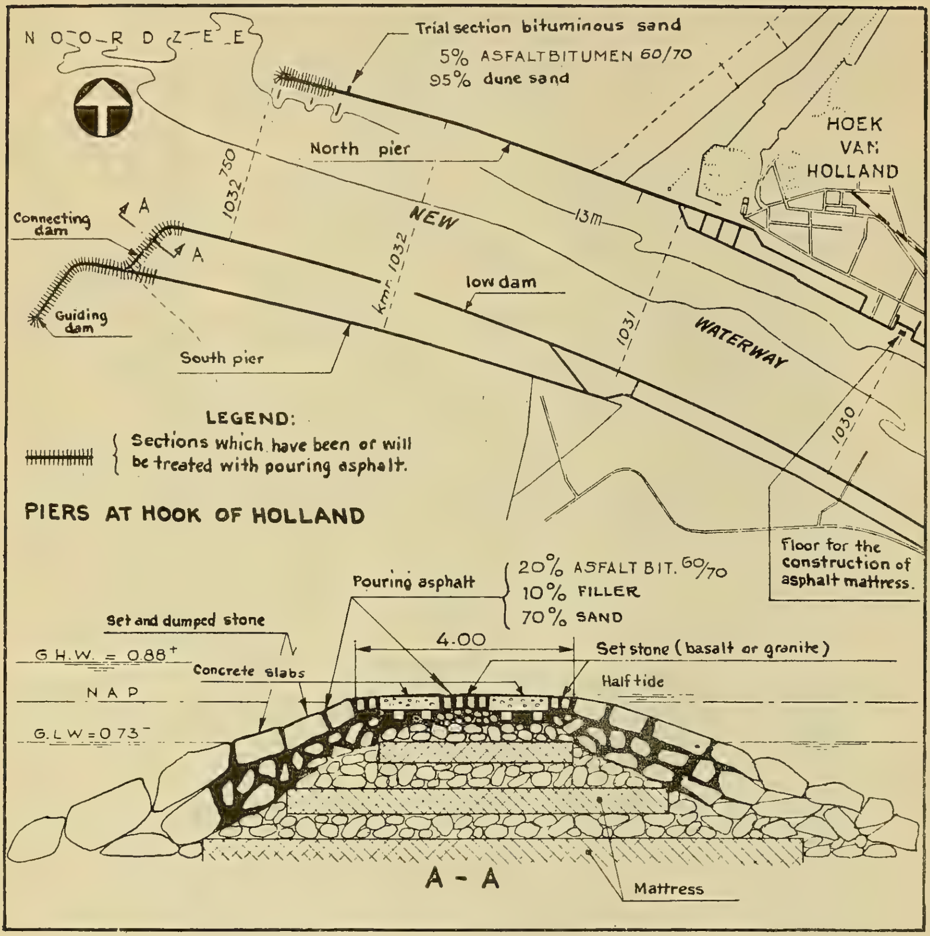


Figure 20. Reinforcement of the offshore sections of the piers at Hook of Holland.

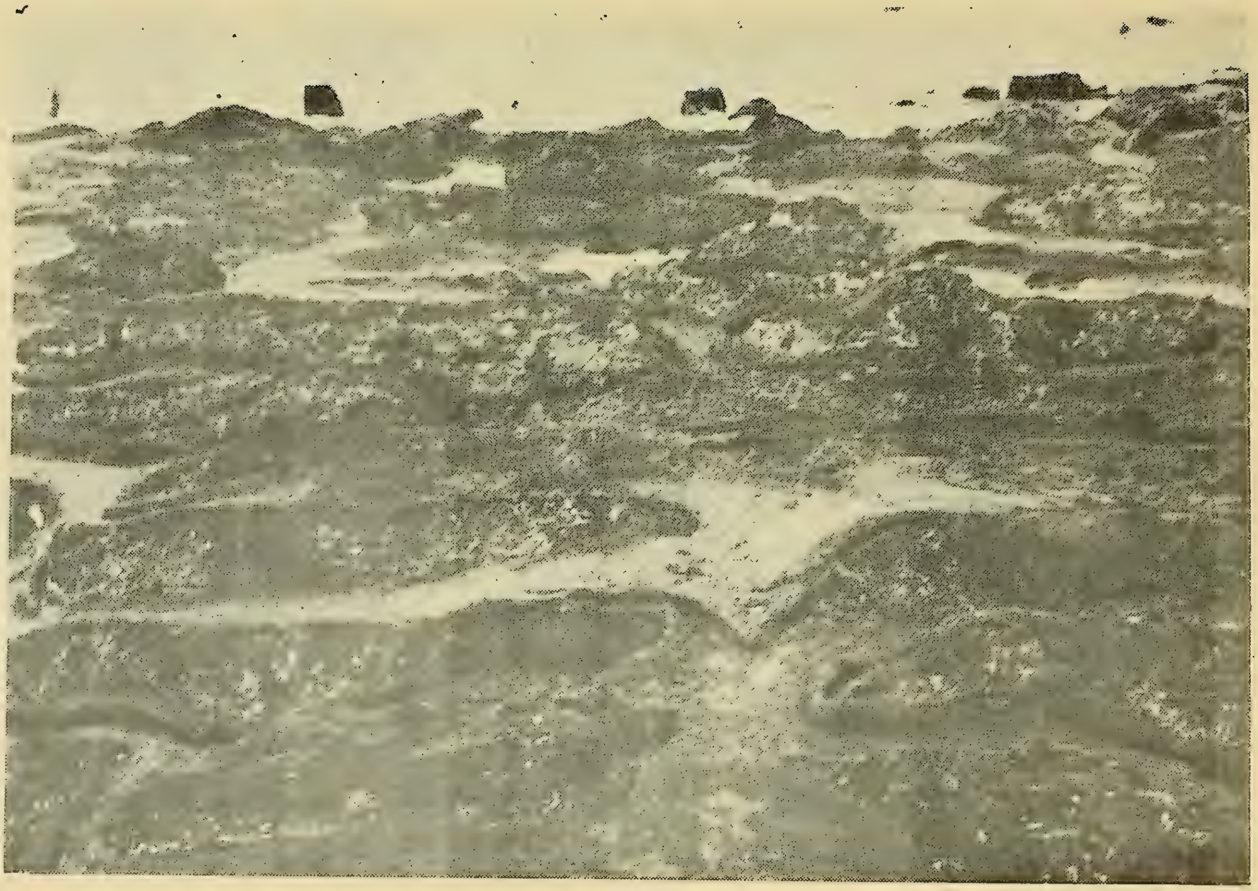


Figure 21. End of the north pier at Hook of Holland. Part of heavily attached stone protection with grouting applied in autumn of 1947.

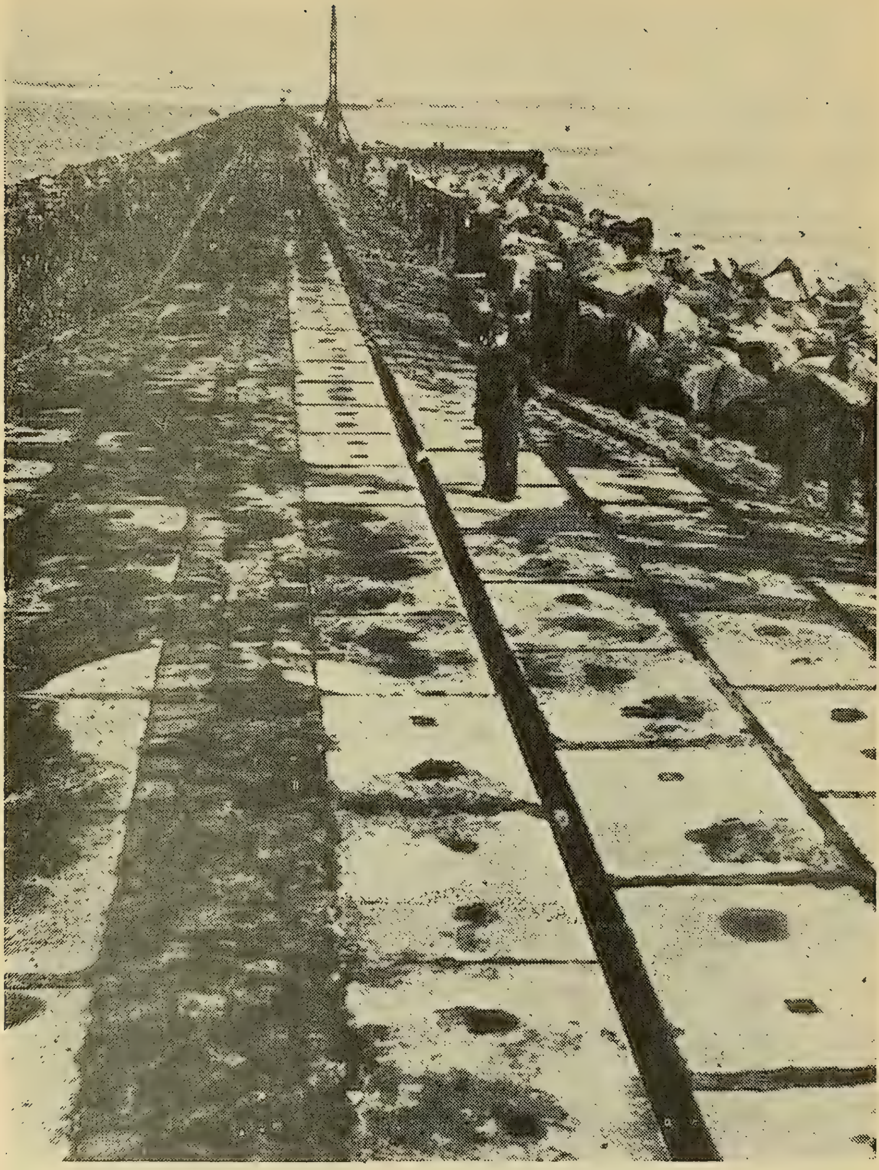


Figure 22. North pier, Hook of Holland. The conversion of the railroad track.into track for trucks, consisting of concrete blocks and set stone with asphalt grouting carried out in 1948. The gutler to carry the lighthouse cable is in the center. In the foreground is the wide section near the foot of the lighthouse.

Figure 23. North pier, Hook of Holland. Damage caused by a gale in September 1948 to a section of the track which was completed, but where the asphalt grouting had not been applied. In the background the track is undamaged where provided with asphalt grouting.

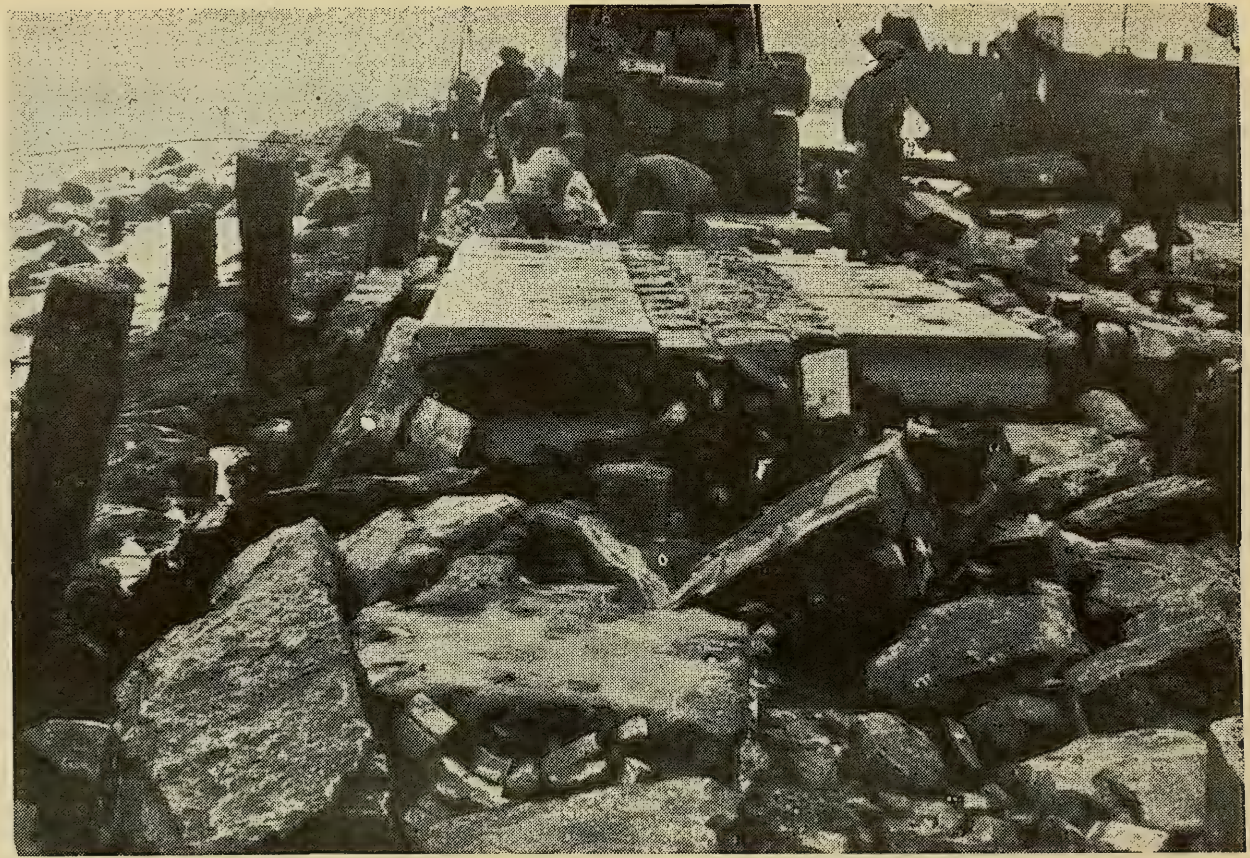


Figure 24. Connecting dam near south pier at Hook of Holland. Damage caused by gale in April 1949 on a short section which was almost completed, but where the sides of the track had not yet been grouted.

Figure 25. The north section of the connecting dam at Hook of Holland which was damaged and partially destroyed by high tides. Picture taken in August 1947 at half tide.

Figure 26. Same view as Figure 25 after reconstruction in winter of 1948-49 and asphalt grouting applied. Picture taken during low water immediately after the heavy gale of 1 March 1949. At the right big boulders from the submarine slope which were carried over the dam by the heavy ground swell.

of 1949. It is proposed to finish this work in 1950. The asphalt mortar contained 20% asphaltic bitumen 60/70, 10% filler and 70% dune sand. Before the pouring of the mortar, the top surfaces of the stones were wetted to prevent the asphalt from sticking to surfaces from which it would be removed by wave action after some time in any event.

The following quantities of pouring asphalt were used successively: between the basalt set stones and in the road on the North pier 250 kg/m²; between the heavy set stones (100-500 kg) 900 kg/m²; in the dumped stone (500-3000 kg) around the head of the pier 1000-1500 kg/m². The greatest care was taken to fill the voids completely in order to avoid cavities in which air and water pressure might develop. The new construction withstood the gale of 1 March 1949 excellently.

In order to test the resistance of bituminous sand against wave action, part of the dumped stone of a mattress which was situated in the breaker zone north of the North pier, was replaced by bituminous sand consisting of 5% asphaltic bitumen 60/70 and 95% dune sand. In total 225 m³ of the mixture was dumped on the mattress.

It was learned from periodic soundings during more than a year, that the bituminous sand is gradually disintegrated by the wave action. Furthermore a great number of the American stone borer (Petricola pholadiformis) was found in a sample.

In cooperation with the Rijkswegenbouwlaboratorium a container with samples of bituminous sand of various composition has been placed in the breaker zone so that it may be determined which asphalt content is required to make the material proof against the stone borer.

f. Miscellaneous asphalt works on beach groins and harbour jetties. - In 1948 the dumped stone deposits around the outer sections of the Scheveningen harbour jetties were filled with pouring asphalt. In the same year a beach groin built as a sand mound covered with bituminous sand, was constructed in the 1944 gap in the Westkapel sea dike (Figure 2). In 1949 the reinforcement of dumped stone deposits of certain beach groins of Delfland, and the province of North Holland (south of Kamperduin and along the North sea shore of Vlieland will be carried out.

Conclusion

In hydraulic works as in road construction, synthetic asphalt has rapidly become a useful building material. Thanks to the extensive research work of various laboratories, it is now possible to produce asphalt mixtures with properties to

suit the type of work.

In many instances, the use of asphalt mixtures in hydraulic works has not led to favorable results, which seems only natural in view of the fact that this material originally was unsuitable for this purpose. However these failures should not make us look for other methods instantly. The age old science of hydraulic engineering has produced many an ingenious method to fight the water which undoubtedly may be used once more in the application of asphalt in hydraulic works. Only if this proves to be impossible will new working methods have to be found to solve the problem.

Certainly the motto of J.F.W. Conrad, which he used in 1864 for his prize-winning contribution to a contest regarding the improvement of the Hondsbossche sea dike, still holds a truth: "Don't condemn the old because it's old, nor the new because it's new."

* * * * * *

BEACH EROSION STUDIES

The principal types of beach erosion reports of studies at specific localities are the following:

a. Cooperative studies (authorization by the Chief of Engineers in accordance with Section 2, River and Harbor Act approved on 3 July 1930).

b. Preliminary examinations and surveys (Congressional authorization by reference to locality by name).

c. Reports on shore line changes which may result from improvements of the entrances at the mouths of rivers and inlets (Section 5, Public Law No. 409, 74th Congress).

d. Reports on shore protection of Federal property (authorization by the Chief of Engineers).

Of these types of studies, cooperative beach erosion studies are the type most frequently made when a community desires investigation of its particular problem. As these studies have greater general interest, information concerning studies of specific localities contained in these quarterly bulletins will be confined to cooperative studies. Information about other types of studies can be obtained upon inquiry to this office.

Cooperative studies of beach erosion are studies made by the Corps of Engineers in cooperation with appropriate agencies of the various States by authority of Section 2, of the River and Harbor Act approved 3 July 1930. By executive ruling the cost of these studies is divided equally between the United States and the cooperating agency. Information concerning the initiation of a cooperative study may be obtained from any District Engineer of the Corps of Engineers. After a report on a cooperative study has been transmitted to Congress, a summary thereof is included in the next issue of this bulletin. A list of cooperative studies now in progress follows:

COOPERATIVE BEACH EROSION STUDIES IN PROGRESS

NEW HAMPSHIRE

HAMPTON BEACH. Cooperative Agency: New Hampshire Shore and Beach Preservation and Development Commission.

Problem: To determine the best method of preventing further erosion and of stabilizing and restoring the beaches, also to determine the extent of Federal aid in any proposed plans of protection and improvement.

MASSACHUSETTS

PEMBERTON POINT TO GURNET POINT. Cooperating Agency: Department of Public Works, Commonwealth of Massachusetts.

Problem: To determine the best methods of shore protection, prevention of further erosion and improvement of beaches, and specifically to develop plans for protection of Crescent Beach, The Glades, North Scituate Beach and Brant Rock.

CONNECTICUT

STATE OF CONNECTICUT. Cooperating Agency: State of Connecticut (Acting through the Flood Control and Water Policy Commission).

Problem: To determine the most suitable methods of stabilizing and improving the shore line. Sections of the coast will be studied in order of priority as requested by the cooperating agency until the entire coast is included.

NEW YORK

JONES BEACH. Cooperating Agency: Long Island State Parks Commission

Problem: To determine behavior of the shore during a 12-month cycle, including study of littoral drift, wave refraction and movement of artificial sand supply between Fire Island and Jones Inlets.

NEW JERSEY

OCEAN CITY. Cooperating Agency: City of Ocean City.

Problem: To determine the causes of erosion or accretion and the effect of previously constructed groins and structures, and to recommend remedial measures to prevent further erosion and to restore the beaches.

VIRGINIA

VIRGINIA BEACH. Cooperating Agency: Town of Virginia Beach.

Problem: To determine the methods for the improvement and protection of the beach and existing concrete sea wall.

SOUTH CAROLINA

STATE OF SOUTH CAROLINA. Cooperating Agency: State Highway Department.

Problem: To determine the best method of preventing erosion, stabilizing and improving the beaches.

FLORIDA

PINELLAS COUNTY. Cooperating Agency: Board of County Commissioners.

Problem: To determine the best methods of preventing further recession of the gulf shore line, stabilizing the gulf shores of certain passes, and widening certain beaches within the study area.

LOUISIANA

LAKE PONCHARTRAIN. Cooperating Agency: Board of Levee Commissioners, Orleans Levee District.

Problem: To determine the best method of effecting necessary repairs to the existing sea wall and the desirability of building an artificial beach to provide protection to the wall and also to provide additional recreational beach area.

TEXAS

GALVESTON COUNTY. Cooperating Agency: County Commissioners Court of Galveston County.

Problem: To determine the best method of providing a permanent beach and the necessity for further protection or extending the sea wall within the area bounded by the Galveston South Jetty and Eight Mile Road.

To determine the most practicable and economical method of preventing or retarding bank recession on the shore of Galveston Bay between April Fool Point and Kemah.

CALIFORNIA

STATE OF CALIFORNIA. Cooperating Agency. Division of Beaches and Parks, State of California.

Problem: To conduct a study of the problems of beach erosion and shore protection along the entire coast of California. The initial studies are to be made in the Ventura-Port Huneme area, the Santa Monica area and the Santa Cruz area.

WISCONSIN

RACINE COUNTY. Cooperating Agency: Racine County.

Problem: To prevent erosion by waves and currents, and to determine the most suitable methods for protection, restoration and development of beaches.

KENOSHA. Cooperating Agency, City of Kenosha.

Problem: To determine the best method of shore protection and beach erosion control.

OHIO

STATE OF OHIO. Cooperating Agency: State of Ohio (Acting through the Superintendent of Public Works).

Problem: To determine the best method of preventing further erosion of and stabilizing existing beaches, of restoring and creating new beaches, and appropriate locations for the development of recreational facilities by the State along the Lake Erie shore line.

TERRITORY OF HAWAII

WAIKIKI BEACH:
WAIMEA & HANAPEPE, KAUAI. Cooperating Agency: Board of Harbor Commissioners, Territory of Hawaii.

Problem: To determine the most suitable method of preventing erosion, and of increasing the usable recreational beach area, and to determine the extent of Federal aid in effecting the desired improvement.

* * * * *

BEACH EROSION LITERATURE

There are listed below some recent acquisitions to the Board's library which are considered to be of general interest. Copies of these publications can be obtained on 30-day loan by interested official agencies.

"Refraction of Shallow Water Waves: The combined effect of currents and underwater topography," Trans. American Geophysical Union, Vol. 31, No. 4

This paper gives a solution for determining the refraction effect according to Fermat's Principle for shallow water waves moving in any given distribution of currents and depth. Application is made to an analytic model of an intense rip current and the results are compared to actual rips.

"A Method of Measuring Electrically the Velocity of Fluids," B. Thurlemann

This paper discusses the theory on which an ocean current velocity meter is developed and also discusses the construction of a model instrument based on this theory.

"The Limitations of the Principle of Superposition," Paul R. Heyi, Jour. Washington Academy of Sciences, Vol. 40, No. 11, 1950.

This paper gives a brief discussion of the principle of superposition as applied to harmonic analysis of wave forms. Several allowable and several erroneous applications of the principle are discussed which are of interest to engineers studying wave forms such as wind waves and tides.

"Model and Prototype Stuides for the Design of Sand Traps," R. L. Parshall, July 1950

The hydraulic laboratories at Fort Collins have made various investigations to develop a means for ridding channels of bed load deposits. Out of these investigations come two practical means, namely the vortex tube and the ripple deflector-vortex tube sand traps which are discussed in this paper.

"Turbidity Currents as a Cause of Graded Bedding, Jour. of Geology, Vol. 58, No. 2, March 1950.

This paper discusses the causes and magnitude of particle grading by the action of turbidity currents. Naturally observed grading and artificially produced grading are described.

"Talud limite entre la rotura y la reflexion de las olas: The Critical Slope Between Incipient Breaking and Reflecting of Waves"; Ramon Iribarren Cavanilles, Casto Nogales y Olano, Feb. 1950.

Translation by Waterways Experiment Station and published as Translation No. 50-2, 1950.